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THE ORDINANCE FOR THE PREVENTION OF ELECtrolysis at Norfolk, Va., noted in our issue of Sept. 19, 1901, has at last been adopted with some modifications by the city council. It provides that all companies or persons using or employing electric current must take proper and effective measures to protect the city "water mains, pipes, cables and other structures" from injury by electrolysis; establishes a penalty for failure to do so; provides for electrical surveys to determine whether or not pipes or cables are endangered by electric currents; and stipulates that whenever and wherever such danger exists the electrical companies must bear the expense of trenching to determine the nature and extent of the danger; and also that the electrical companies must not only stop the electrolysis, but also pay for all damages caused thereby. Tests shall be made by an expert selected by the city electrician and the private corporation whose electric current is involved, and in case of disagreement the city engineer will act as an umpire. The whole matter of electrolysis is under the general charge of a board of electrical control. We are indebted to Mr. W. T. Brooke, City Engineer of Norfolk, for a copy of the ordinance.

BIDS FOR PUBLIC ELECTRIC LIGHTING at New Orleans, La., are wanted until March 24, 1902. The successful bidder must construct a plant in accordance with plans and specifications on file with the city engineer. There will be 301 lamps in the underground and 2,299 lamps in the overhead district, besides which there will be required 150,000-K-W. hour service for public buildings. The street lamps will be series alternating enclosed arc, using an amount of electrical energy represented by a current of 6.6 amperes with a difference in potential of not less than 72 volts per lamp, measuring not less than 475 watts at the terminal of each lamp. The contract will be for ten years, at the termination of which period the plant will become the property of the city without further payment than the annual contract price.

THE WATER METER CONTRACT AT CLEVELAND, O., was authorized by the city council on Feb. 17 by a vote of 10 to 6, after a previous temporary defeat. This means that some 11,000 house meters will be set during the present year, after which it is hoped that meters will become so popular as to lead to successive large installations until the 50,000 unmetered services now in existence have all been changed to metered services. Mr. Chas. P. Salen is Director of the Department of Public Improvements and Prof. Edw. W. Bemis is Superintendent of Water-Works. The adoption of meters is also largely due to Mr. Tom Johnson, Mayor of Cleveland.

THE COST OF STREET PAVING AT COLUMBUS, O., according to statements made by Colonel Abbott, a contractor of that city, in an address before an organization of young men, is approximately as follows: Granite block, $4 per sq. yd.; Medina block sandstone, $3.75; asphalt, $2; granite macadam, $2; brick, $1.50; limestone and boulders, 75 cts. each. These figures do not include grading, and are based on local standard specifications and prices.

ARTIFICIAL CLAY, for use in the manufacture of artificial stone, tiles, gutters, etc., is now made in Germany. This substance is made up of sand, chalk, cement, liquid glue and petroleum, intimately mixed in large quantities; a chalklike mass is the result, which is easily molded and made hard by the application of heat. The products have a perfectly even surface and sharp edges; they are fireproof, and resist weather. They are further non-absorbent to moisture, says Mr. R. Guenther, U. S. Consul-General at Frankfort, Germany.

THE MOST SERIOUS RAILWAY ACCIDENT of the week occurred on the Auburn branch of the New York Central & Hudson River Ry. on the evening of Feb. 24. A west-bound passenger train, when rounding a curve near Auburn, N. Y., at fair speed, collided with a steam derrick running wild to Syracuse. As far as known at the present, six men were killed and a number of others injured. It is said that the passenger train had the right of way, and that the responsibility for the wreck rests with the engine crew of the derrick.

AN EARLY MORNING FIRE IN A FIREPROOF hotel caused the death of a score of people in New York city on Feb. 22, 1902. Shortly after 1 a. m. of that day the fire department was called to Fourth Ave. and 34th St., where the 71st Regiment Armory was in flames. This structure was completely destroyed. A high wind was blowing at the time, and sparks from the burning armory were carried across the street and set fire to the Park Avenue Hotel, located on Fourth Ave., diagonally opposite the armory. Before it was discovered that the hotel was burning the fire had spread so that escape or rescue of the people within was very difficult. The excavation for the Rapid Transit subway just at this point set special obstacles in the way of the firemen. Before their work was completed, seventeen persons were suffocated or burned in the rooms and corridors of the hotel, and a large number of others injured; of the latter, two died within a few days. The hotel was built about thirty years ago, and was considered to be fully fireproof, according to the knowledge of that time. It was eight stories high, about 200 ft. square in plan, had five interior staircases and three elevators, but no fire escapes.

THE USE OF AUTOMATIC FIRE SPRINKLERS in the last six years, according to the last annual report of the Boston Manufacturers Mutual Fire Insurance Co., has reduced the actual loss by fire in buildings so protected to less than 1 cent. per $100 of risk carried one year.

ALL BOILERS ON STEAM VESSELS must hereafter be equipped with fusible plugs made of brass and filled with banca metal, according to an order of the government inspectors of steam vessels, at their recent meeting in Washington. The inspectors will not pass a vessel when the boilers are not fitted with fusible plugs, and many styles of plugs now in use will be rejected, only those passing inspection which are approved by the inspectors.

YOKOHAMA TO LIVERPOOL IN 31 DAYS is the record lately made by a consignment of freight. It left Yokohama on Dec. 27 by a Canadian Pacific Railway Co.'s steamer; reached Vancouver—4,283 miles, on Jan. 9; West St. John—7,670 miles, on Jan. 17; and Liverpool—10,391 miles, on Jan. 28.

THE BLOCK SYSTEM is being introduced on the Union Pacific Ry., but no extensive signal plant is being installed, the train-order signals being used to space trains about 10 miles apart.

TEXAN FUEL OIL FOR THE NAVY is the object of a request from the Navy Department for an appropriation of $20,000 with which to conduct tests of the value of this oil as fuel for warships, the tests to be conducted by the Bureau of Steam Engineering. The explanatory note accompanying the letter of submission says that "in general one pound of liquid fuel should evaporate 50% more water than a pound of excellent coal." A Paris item says that the French navy is likely to place enormous orders for Texan petroleum briquettes made under French patents.

GOVERNMENT OWNERSHIP OF THE PACIFIC cable is recommended in the majority report of the House Committee on Interstate and Foreign Commerce. The report says that a Pacific cable is a necessity for the future peace and prosperity of our insular possessions, as these depend largely on the facility of communication. As to government ownership, the committee quotes the action of other nations in this direction. England has expended $100,000,000 in constructing and purchasing cable lines, and France and Germany are following the same policy. The transmission of communication should be held by the State, and its control is an inherent right of the nation. Gen. Greely estimates the cost of a Pacific cable at $10,000,000, with $525,000 for annual maintenance. From a technical and engineering point of view, the practicability of such a cable is placed beyond doubt. A minority report from the same committee opposes government construction and favors the proposal of the Commercial Cable Co. to lay a private cable.

A LARGE FLOATING DOCK FOR BERMUDA was launched at Wallsend, England, on Feb. 8. It is intended for government use and has a lifting capacity of 17,500 tons and an area of 69,000 sq. ft., making it one of the largest floating docks ever constructed.

THE BERLIN ELECTRIC RAILWAY, partly elevated and partly underground, was formally opened to traffic on Feb. 15. The elevated part is on stone or iron piers and sometimes on stone viaducts. Of the total length of line about 27% is on curves, some of them unusually sharp. The main system runs east and west through the German capital, with an underground terminal at the Potsdamer Platz, and with another terminal at the Warshauer bridge. The rolling stock now includes 42 motor cars and 21 trailers, the motors seating 40 people and the trailers 60. The usual headway is 5 minutes. The city of Berlin, under the contract with the operating company, is to receive 2% on yearly receipts of $1,500,000 and ¼% additional on every $250,000 above that amount.

A NOVEL MEASURING MACHINE, giving direct readings on a graduated dial to 1-20,000-in., has recently been placed on the market by the John M. Rogers Boat, Gage and Drill Works, of Gloucester City, N. J. As described in the "American Machinist" of Feb. 13, the machine is made in two sizes, one measuring up to 12 ins. and the other to 24 ins. A system of multiplying levers is used to gage the pressure on the piece being measured, so as to eliminate the personal factor in making measurements.

THE METRIC SYSTEM IN THE UNITED STATES has been reported upon by a special committee of the Franklin Institute, of Philadelphia. The committee recommended the approval of any movement which will promote the universal adoption of the metric system with the least confusion and expense; and it would ask that the national government enact such laws as will insure the adoption of this system of weights and measures as the sole standard in its various departments as rapidly as is consistent with the public service. The recommendation of the committee has yet to be debated and passed upon by the Institute. The committee based its conclusions upon the following conditions: Other nations will not adopt our system; the metric system has inherent advantages in computation; the only objection made to it is that the numeration cannot be continuously subdivided by two, but the same is true of our decimal currency; it believes that such a change of systems could be inaugurated by the government; in workshops, a large proportion of existing tools and gages could be retained until they were gradually superseded, merely designating their nominal dimensions in the nearest metric units.

FRENCH SCHOOLS OF ENGINEERING and of political science are to be established in this country under the control of the French government, in accordance with the suggestion originally made by MM. Millerand and Caillaud, the French ministers of commerce and finance. Two commissioners—M. Maurice de Lagotellerie and M. Lazare Weiller, are now in this country studying our industrial combinations, their formation and operation. These commissioners are also charged with selecting sites for the two schools referred to, and the engineering school is to be located in Pittsburg and that of political science in New York. It is the intention of the French government to select the best pupils in the French schools of commerce and to send them to the French schools in the United States to complete their industrial and commercial training after American methods. It is proposed to send five pupils in each of the departments of civil, mining, mechanical and electrical engineering to the Pittsburg school for a two or three years' course.

JAPANESE-UNITED STATES COMMERCE, says the U. S. Bureau of Statistics, has grown from less than 6% of the total importation into Japan in 1881, to 22% in 1900; while in the same period Great Britain's share in these imports has fallen in the same period from 52% to 25%. In 1900 the total exports of Japan amounted to 198,000,000 yen, and of this sum 52,566,000 yen went to the United States; the exports consisting largely of raw and manufactured silk, mats and rice. The imports from the United States to Japan, in 1900, amounted to 62,761,196 yen, the largest items being illuminating oil, manufactures of iron and steel, raw cotton, breadstuffs, paper, tobacco, etc.

WATER TOWER AT WILMINGTON, DEL., WITH STEEL AND CONCRETE ROOF.

A novel form of roof is used to cover the masonry-enclosed stand-pipe at Wilmington, Del., locally known as the Rockford water tower. The roof is supported on steel trusses, and has an outer sheathing of copper tiles, but the body of

Fig. 1. View of Rockford Water Tower, Wilmington, Del.

the roof covering is of 3-in., No. 10 expanded metal, embedded in the bottom of cinder concrete slabs 2½ ins. thick.

The stand-pipe has a capacity of about 560,000 gallons, and is supported on steel floor beams,

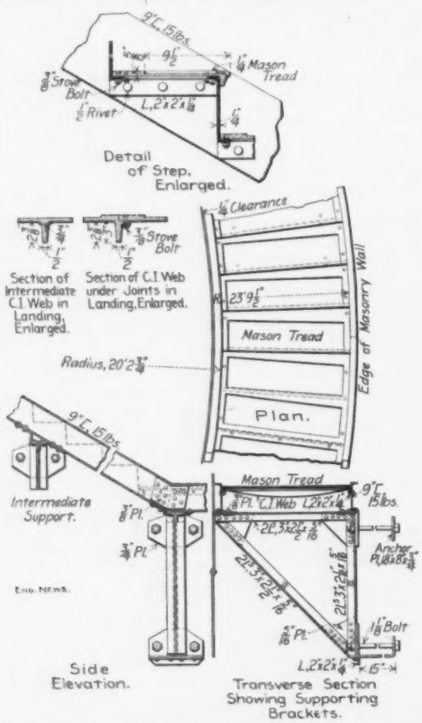

Detail of Step, Enlarged.
Section of Intermediate C.I. Web in Landing, Enlarged.
Section of C.I. Web under Joints in Landing, Enlarged.
Mason Tread
Edge of Masonry Wall
Plan.
Intermediate Support.
Side Elevation.
Transverse Section Showing Supporting Brackets.

Fig. 3. Details of Winding Stairway in Wilmington Water Tower.

The dimensions of the stand-pipe and tower, and the method of joining the bottom and sides of the stand-pipe are shown by Fig. 2. The tower is built of rough-face granite masonry, and is 56 × 123 ft. The inside of the tower is lined with white enameled brick. The space between the masonry wall and the steel stand-pipe is utilized for a stairway, which gives access to the observatory, commanding a fine view of Wilmington and the surrounding country. Details of the stairway are shown by Fig. 3. The observatory floor is of the ordinary type of expanded metal and steel construction.

The tower cost about $76,000. It was built by the Wilmington water board, of which Mr. Jos. A. Bond is superintendent, to supply a high-level section of the city. The plans for the structure were made by Mr. Theo. A. Leisen, of Wilmington, Del., and Baker & Dallett, of Philadelphia, Pa., were the architects for the tower. The concrete and expanded metal work was executed by Merritt & Co., of 1024 Ridge Ave., Philadelphia.

THE RECENT EPIDEMIC OF TYPHOID FEVER AT BARABOO, WIS., AND ITS PROBABLE RELATION TO THE WATER SUPPLY.

By W. G. Kirchoffer, C. E.*

The city of Baraboo has a population of about 6,000 (5,751 in June, 1900.—Ed.), and is situated on the Chicago & Northwestern Ry., 37 miles north of Madison. The city lies between two quartzite ranges, about five miles apart, having a general east and westward direction and a height of 400 to 800 ft. above the valley below, through which flows the Baraboo River. The ranges of quartzite are crossed in nearly a north and south direction, just west of the city, by the great moraine of the last glacial epoch in Wisconsin.

The surface of the valley is, therefore, generally drift, covered with a few outcrops of Potsdam sandstone. Along the flanks of the ranges are numerous outcrops of the quartzite. The drift consists principally of sand and gravel, with some sandy clay and boulders. The sides of the valley being very steep and the soil porous, afford good conditions for the formation of springs, which abound in all parts of the valley. It is from some of the largest of these springs that the supply of water is obtained. This supply was developed and the water-works constructed in 1887 by Moffet, Hodgkins & Clark, who operated the plant for a few years and then failed. Since then the plant has been operated by the second mortgage bondholders, and is now in the hands of the receiver for the first mortgage bondholders. Since the receiver took charge of the works the plant and source of supply have been improved at an expense of about $10,000. The source of supply now consists of a round iron reservoir or well, 20 ft. in diameter by 30 ft. deep, constructed around a spring; a similar round well of stone, 30 ft. in diameter by 28 ft. deep; a long stone wall or gallery about 20 × 110 ft. in plan, and 8 ft. deep, with stone laid up dry; and an artesian well 8 ins. in diameter, with 14 ft. of Cook point. This well discharges 110 gallons per minute into the stone well, or well No. 2, as shown upon the map, Fig. 1. The flow is secured from the drift at a depth of 87 ft., a test well having previously been sunk to a depth of 183 ft., where quartzite was encountered. The artesian well was sunk in the summer of 1899. The water from the gallery and well No. 2 is drawn to well No. 1 by siphons; from well No. 1 the supply is drawn through a 14-in. suction pipe to the pumps.

The pumping plant consists of two 1,000,000-gallon duplex water-power pumps, one compound Worthington duplex steam pump of about the same capacity, two turbine water wheels, and a 75-HP. boiler. The suction pipe from the wells as it enters the building crosses one of the flumes to the water wheels about 1 ft. above the bottom of the flume, so the water of the flume is in contact with the outside of the suction pipe at all times.

The water from wells Nos. 1 and 2 has always been of fair quality, though containing a small quantity of iron. During the time of floods on the river the wells have been occasionally overflowed, but that has not caused serious trouble. The water from the long well or gallery, as might be expected from its construction, has always given the worst analysis. The water from the artesian well has always been of fine quality, but has always carried a large amount of iron.

The first case of typhoid fever occurred at the

*Consulting Engineer, Baraboo, Wis.

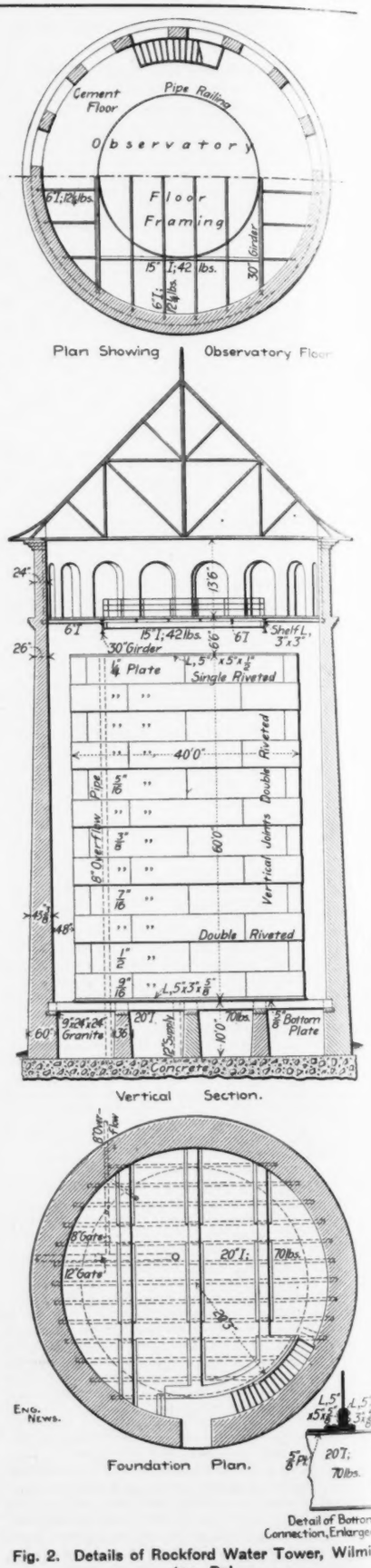


Fig. 2. Details of Rockford Water Tower, Wilmington, Del.

point A, Fig. 1, in March of 1901. The next case and the first one recorded upon the diagram, Fig. 2, occurred at the point C on the map. The latter patient was a plumber, and at the time he was taken sick was reconstructing a sewer in the central portion of the city. The main sewers discharge into the river at locations shown on the map by stars.

As soon as it was evident that an epidemic of fever was upon the city, the authorities proceeded at once to determine, if possible, the cause. After

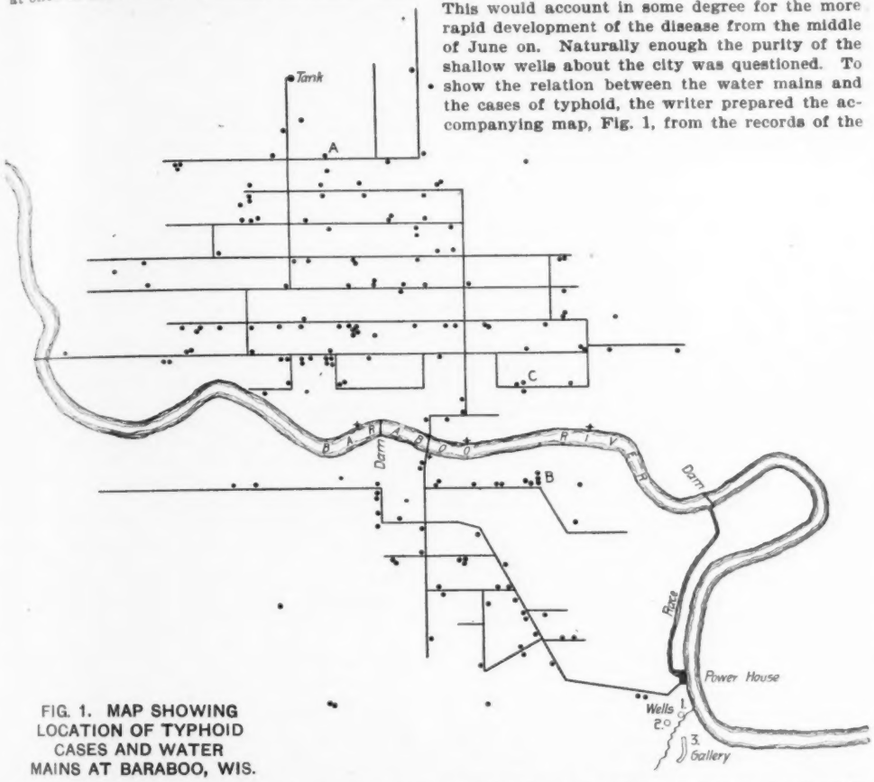


FIG. 1. MAP SHOWING LOCATION OF TYPHOID CASES AND WATER MAINS AT BARABOO, WIS.

several analyses of the water from all of the wells and from the service pipes uptown, it was decided that the water was at least partly at fault. The water company promptly went to work to ascertain the pollution of their supply. Tests for percolation from the creek to well No. 1 showed negative results. Tests for leakage through the valve shutting off the connection with the race also gave negative results.

For some time previous to the outbreak of fever it had been noticed that when the steam pump was running a large quantity of air was pumped and that the pump pounded to such an extent that the rear end of its foundation was pulled loose. When the water-power pumps were running this difficulty was not noticed, since the motion was slower. Well No. 1, from which all of the water is pumped, was cleaned by means of a centrifugal pump, and the foot valve of the main suction was removed and examined. It was found that five of the eleven $\frac{5}{8}$-in. valves and one of the two 2-in. valves were sealed shut with iron rust. This was no doubt the cause for the pounding of the pump and the presence of air in the pipe, which was drawn in through an unknown opening on account of the heavy suction due to the friction of the water passing through the greatly reduced area of the foot-valve. The natural lift was about 13 ft. Upon further investigation there were found five small leaks in the suction pipe where it passed through the flume. The joints of this pipe were the ordinary bell and spigot joints caulked with lead. As will be seen on the diagram, Fig. 2, the stoppage of these leaks on Aug. 12, had a marked effect upon the development of the disease. No doubt the warning issued to the public on July 30 also had some effect on the spread of the fever, though it is well known to the writer that there were parties using the water after this warning. It will be noted by the diagram that the fever began in a very mild form about the first of May, and continued so until June 17, when it began to develop rapidly.

Ordinarily the steam pump is used only when the water in the race is too low to run the water wheels. This year the steam pump was put in operation to a considerable extent during the month of May. On account of the loose foundation above referred to a larger quantity of water from the race was no doubt drawn in through the leaks, thus causing a greater pollution than usual. This would account in some degree for the more rapid development of the disease from the middle of June on. Naturally enough the purity of the shallow wells about the city was questioned. To show the relation between the water mains and the cases of typhoid, the writer prepared the accompanying map, Fig. 1, from the records of the health officer. It will be noted that all but twelve of the cases are situated along water mains. Many people, not water takers, claimed to be using city water, and there were about 35 cases out of the 184 reported that claimed to be using either rain or well water.

The only location in the city where it would

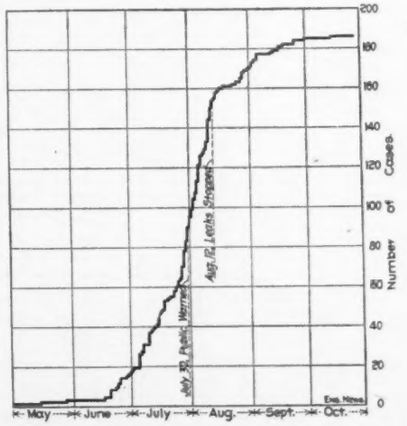


Fig. 2. Diagram Showing the Development of Typhoid Fever at Baraboo, Wis., in 1901.

seem probable that the shallow wells were polluted was at the point B, Fig. 1, where four members of one family had the disease. The total number of deaths due to typhoid, including, in a few cases, various complications, has not exceeded 15. While we have not prima facie evidence that the disease was caused and spread by the city water, nor that we would not have had an epidemic of the fever had we not have had the conditions as they existed at the pumping station, yet it would appear from the facts here submitted that the real cause of the disease is indirectly traceable to the artesian well. The large amount of iron in this water caused the stoppage of the foot-valve. This, with the ensuing chain of circumstances, caused the pollution of the water. While the water company has been condemned in pretty strong terms by some of the citizens of this city, yet it cannot be shown that the company's servants wilfully pumped polluted water into the mains; in fact there is no evidence to show that they have since the present management took charge of the works.

In conclusion, it appears to me that the chief lesson to be learned from this experience is that it is not safe to run a pure water main through polluted water with no other protection than a lead joint, especially when heavy machinery is apt to be jarring and pulling upon such a pipe. In addition it is always best, even at a considerable expense, to maintain pumps in proper working order. If the company at Baraboo could be censured at all, it is for not looking into the trouble existing with the steam pump before they did. A large quantity of iron in water is bound to make trouble somewhere sooner or later, and when possible it would be preferable to avoid the use of such water.

THE COMPARATIVE ECONOMY OF GAS ENGINE AND ELECTRIC MOTOR FOR SMALL POWERS.

The gas engine and the electric motor have for many years been in competition with each other for small power installations in towns and cities where both gas and electric current are generally available. Wherever the power is required intermittently, and frequent stopping and starting is necessary, or where the size of the motor required is very small, the electric motor is, of course, by far the better; but where a motor is run steadily all day, the lower cost of operating the gas engines in many cases more than offsets the other advantages of the electric motor.

An excellent opportunity to compare the economy of the two systems is furnished by the experience of the printing office in which the composition on Engineering News is done, and we are able to present the following comparative figures through the courtesy of its owner and manager, Mr. Charles J. O'Brien.

Power is used chiefly to run linotype machines and small job presses. At present two motors are in use, a 15-HP. gas engine and a 25-HP. Crocker-Wheeler electric motor, the latter installed sometime in 1901. The average load carried is only about 10 HP.; but occasionally a much heavier load is thrown on, necessitating a large-capacity motor. City illuminating gas is used, costing $1.05 per M. cu. ft. Electric current is taken from the Edison mains, and the price is 10 cts. per E. HP.-hour, subject to a discount of 50% and 5%, making the net price 4.75 cts. per HP.-hour.

A comparison has been made between the two months, Oct. 9 to Dec. 11, 1900, when the gas engine alone was used, and the two months, Oct. 4 to Dec. 5, 1901, when the electric motor alone was used. The results are summarized in the following table:

Kind of motor.	Gas engine.	Electric motor.
Period of use.....	Oct. 9 to Dec. 11, 1900.	Oct. 4 to Dec. 5, 1901.
Total hours machines were run	697	686
Cost of gas or current used....	$106.68	$250.30
Average cost per hour.....	15.3 cts.	36.5 cts.

THE IMPORTS AND EXPORTS OF COAL for 1900 and 1901 are reported by the U. S. Bureau of Statistics for 11 months of each year ending in November. The total imports—largely to the Pacific Coast—were 1,757,561 tons in 1900 and 1,745,636 tons in 1901 for 11 months. Of this total in 1901 1,321,004 tons came from British North America. For the same time the exports of anthracite coal from the United States to all countries was 1,495,327 and 1,889,438 tons for 1900 and 1901, and of bituminous coal 5,649,712 and 5,043,221 tons, respectively. Of the total export of coal 4,794,431 tons went to British North America in 1901. The next largest buyers were Mexico, 520,568 tons; Cuba, 336,855 tons; other West Indies and Bermuda, 323,027 tons; France, 217,024 tons; other Europe, 302,142 tons; South America took 111,713 tons.

A NON-CONTINUOUS SWING BRIDGE.

Swing bridges, as ordinarily constructed, consist of trusses or girders which, when open, rest on and are balanced over a drum on the middle pier, and which are made continuous or partially continuous over their supports when closed by inserting supports under the free ends.

If these supports merely touched the ends without exerting any uplift, the entire dead load would still rest upon the center pier, a balanced live load would be distributed to the three piers in certain proportions due to the flexibility of the trusses, and a live load on one arm only would make the far end "kick up." To prevent this the end supports are generally made to raise the ends far enough to neutralize this uplift from the unbalanced live load; under this condition the dead load on the center pier is relieved by the amount of uplift exerted at the ends, and a balanced live load is distributed as before.

If now the piers settle unequally there will be a readjustment of the proportions of loads coming upon the supports, and consequently different stresses will be produced in the trusses or girders. If the center pier settles and the end piers remain at the same level as before, the tendency will be towards the condition of two simple spans. If one or both the outer piers settle, the tendency will be towards relieving the end uplifts and producing the condition of a "tipper;" in other words, the condition of loading on the arms lies somewhere between the two possible extremes of two simply-supported independent spans, or of two cantilever arms balanced over the center pier. When the structure is new it may be assumed to act in accord with the assumptions of the designer, but after years of service the conditions change, and then will appear to the engineer who has been called upon to report on the efficiency of the structure in question, how hopelessly indeterminate are the stresses that may be produced by any given loading.

In railroad bridges a further complication is involved by the use of rail lifts. The necessity for exact alinement of the rails on the bridge and abutments has brought into general use a grooved bearing plate resting on the abutments and supporting in common the ends of the rails on bridge and abutments. Now, as the end-lift is withdrawn the ends of the bridge sag down, leaving the ends of the rails resting in their bearings on the abutments. Since the bridge cannot be turned while the rails are in this position, the end sections of the rails on the bridge are left free to pivot in such a way that by means of a system of levers they can be raised clear.

Attempts to simplify the operating machinery of swing bridges, and to improve the resulting condition and operation of the structure date back a considerable number of years. Prior to 1876, the Keystone Bridge Co. had constructed a bridge after the designs of Mr. J. H. Linville, across Raritan Bay, the draw span of which was lifted bodily, by hydraulic rams placed under the center support, to clear the end latches before the bridge was turned.* It is not stated whether the arrangement of the trusses was such as to cause the two side arms to act as simple spans when the bridge was closed. This latter feature, that of making the bridge a "discontinuous" structure when closed, appears† to have been first suggested by Mr. Chas. Shaler Smith in 1880. The object might be attained, it is evident, by slacking off the center top chord sections, through suitable mechanism, after the bridge had been turned to "closed."

A scheme somewhat related to the preceding is that of raising the two side arms by rotating

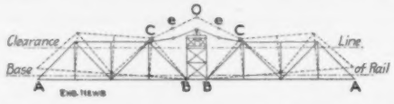


Fig. 1. Diagram of Worthington Swing Bridge.

them about their inner end pins, for instance, by drawing up the top chord bars in the sections immediately at either side of the center. This was stated by Mr. C. A. P. Turner (Eng. News, May 19, 1892) to have been done in the case of some early wooden Howe truss bridges built in the vicinity of Boston, the top chord bars being drawn in by a system of cams and levers. A development of this idea may be seen in a design by Messrs. C. A. P. Turner and P. A. Warren, shown in Eng. News April 16, 1892. In this design the central tower or frame was made A-shaped, and at its top was arranged at either side of the bridge a vertical screw by which the pin connecting the top chord eye-bars running to the inner hips of the side arms could be raised or lowered. This toggle-like action raised or lowered the ends of the side arms of the bridge by rotating them about their inner bottom chord end pins. By raising them in this way, the ends of the arms cleared their supports so that the bridge could be turned open. It was thought desirable to have the two arms of the bridge discontinuous when the bridge was closed and the two arms lowered to their supports; to accomplish this the top chord eye-bars taking on to the center pin had their holes for this pin slightly elongated or slotted. This allowed a free deflection of one arm and its inner hip without affecting the other; the arms, therefore, acted as simple spans when the bridge was closed.‡

All of these attempts and designs seem to have involved a certain amount of complication and faults of mechanism that prevented their general adoption. Quite recently, Mr. Chas. Worthington, M. Am. Soc. C. E., of Pittsburg, Pa., has designed and patented a new form of swing bridge, similar in its general features to that proposed by Messrs. Turner and Warren above referred to, but improved so as to meet all objections hitherto urged against this type of bridge. The nature and operation of Mr. Worthington's bridge will be most readily understood by reference to Fig. 1, which is a diagram of the truss and the essential operating parts, with their motion shown to an exaggerated scale. As shown in this sketch, the top chord eye-bars C O which connect the inner hips C with the top of the tower, are each hinged and pin-jointed in the middle, at points c, and roll to a common pin, O, immediately above the center of the tower. This pin O slides in a vertical guide-slot in the small A-frame fixed on the top of the tower, and may be raised or lowered by means of screws.

The position of the bridge, when the latter is in its normal or closed condition, is as indicated by the full lines; the outer ends, A, of the arms rest on the abutments, the hinged top chord bars are lowered by means of the screws, so that their middle hinges, c, sag below the straight line, and the two arms of the bridge in consequence act as single spans. Any deflection of either arm with the accompanying motion of point C is allowed for by the sag in bars C O; loads on one arm cannot react on the other, and all kicking-up or hammering is avoided.

When the bridge is to be swung, the screws acting on pin O are first set in operation; in gradually raising O they first take the sag out of bars C O, then by pulling on these bars they rotate points C about B until the abutment pressures at A are reduced to zero, and finally, continuing this operation, they raise points A clear of their supports any required distance. This raising is made only enough to clear the end supports, and the bridge is then rotated in the usual manner. The raised position is shown in Fig. 1 by the dotted lines.

The turntable, drum, supporting girders, etc., are for simplicity not shown in Fig. 1; they may be arranged in the usual way below the inner end pins at points B. The style of tower shown, a rectangular-braced tower, with small A-frame on top to carry the slotted guide-plates, is not essential to the design. As shown, it presents a convenient arrangement where it is desired to mount the operator's house with the machinery in the center of the bridge, above the clearance line of roadway. An A-shaped center frame supporting the guide-plates may be used if desired, instead of the rectangular tower, without altering the other parts of the construction.

On the lines indicated in the preceding, Mr. Worthington has worked out the complete design for a 256-ft. single-track railroad draw-span; the

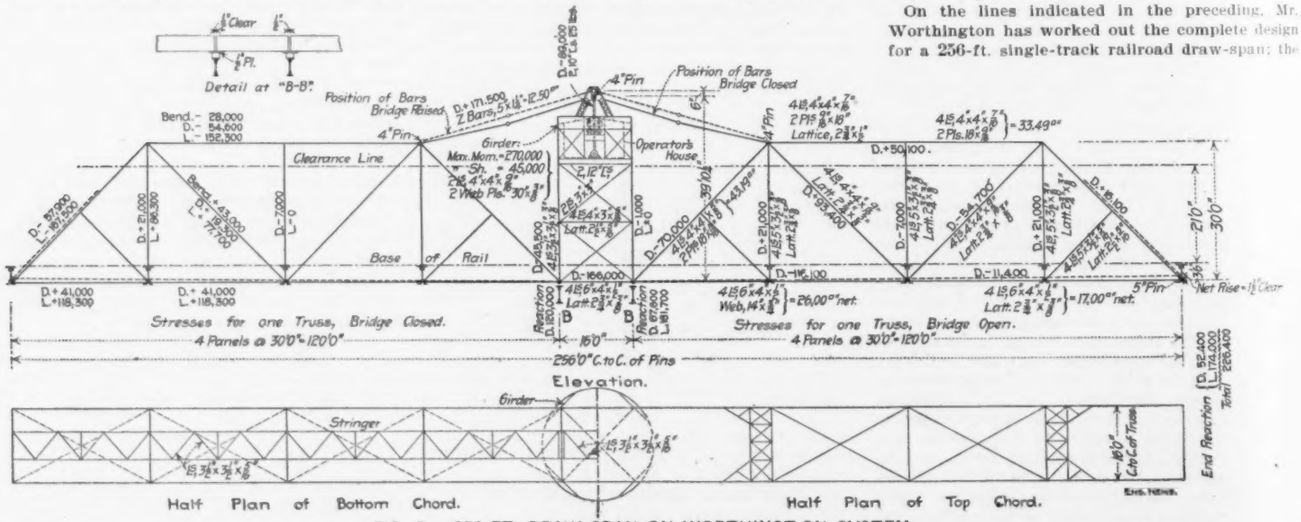


FIG. 2. 256-FT. DRAW SPAN ON WORTHINGT ON SYSTEM.

lifting machinery has been designed in detail, and all necessary calculations made as to power required, time of operating, etc. A diagram of the bridge, giving stresses and sections, is shown in Fig. 2, omitting, however, drum, wheels, etc. In this connection it will be of interest to note that the sizes of truss members for each arm are nearly the same at corresponding points on each side of the center line of the arm; if this structure

*Eng. News, June 16, 1892, letter of F. C. Osborne.
†Letter of J. W. Schaub, Eng. News, May 19, 1892.
‡A design quite similar to this one, but operating by toggle-lever instead of by screw, had been previously proposed by Mr. M. J. Patterson; see Eng. News, May 5, 1892.

were to be of the ordinary, continuous type, the outer ends of each arm would be about the same as those shown, but from about the center of each arm to the center of bridge, the truss members would be considerably heavier than those shown. Thus is affected a considerable saving in material without sacrificing anything of strength. The cross-girders at the center of the bridge are shown at B B, these girders also acting as floor beams at these points. The stringers, which are framed into all the other floor beams, rest upon blocks on the top flanges of these cross-girders, and a clearance space of ½-in. is provided between adjacent stringers over the girders to allow for the motion due to the lifting of the two arms of the bridge. This clearance space is indicated in the small view marked "Section at a-a."

Fig. 3 shows in detail the arrangement of the top of the tower and the mechanism for lifting the pin connecting the top bars. A heavy girder C connects the tops of the tower posts on either side and supports, by means of the framework shown, the seats B for the lifting screws; the latter are held in alinement by an additional bearing engaging their top ends and attached to girder C. Working on the screws S are cast-steel trunnion nuts N, pivoted in the short girder G, and supporting the latter. Rising from the girder G at its center on each side of the bridge is the strut T, which carries at its upper end the main center pin P. This latter is the pin which connects the top chord eye-bars E; it slides in guide-slots cut in plates attached to the triangular frame A. The full lines show the pin and the eye-bars at the lower position of their travel; the dotted lines show them at the upper position. It will be noticed that any unbalanced loading of arms while swinging, such as that caused by wind, unequal levels of end supports, etc., will cause pin P to bear against the guides, but will produce no tendency for the screws to bind in the trunnions. The frames A, which rest on the deep box girders C at either side of the tower are held in position by the transverse brace frames F.

In the cross-section at the right in Fig. 3, the main gear shaft M is shown. This is driven directly from the engine by the large gear N at the center, and in turn drives, through bevel gears at its end acting on short vertical shafts, the pinion O, which meshes into the gears R keyed to the screws S. The shafting and gearing for operating the turntable are not shown; vertical shafts for this purpose may be placed as desired, and driven by another gear shaft and clutch coupling from the same engine or from a separate engine or an electric motor.

It will be noted that in Fig. 3 the maximum lift of the center pin is given as 12 ins. This is intended to cover all contingencies; calculation of the required lift shows it to be ample. The motion of the pin P consists of three parts: (1) that necessary to take all slack out of bars E, (2) that necessary to reduce the pressure of the ends of the bridge on the abutments to zero, (3) that necessary to lift the ends a sufficient clear distance above the supports. Of these (1) is found to be, for a maximum slack in the bars of 9-in. deflection from a straight line, 1 13-32-ins.; (2) is found, by calculating the distortion of the truss when its ends are unsupported, to be 2 ins.; (3) is found, for a clear lift of 1½ ins. at the ends, to be 1 25-32 ins. The sum of these is 5 3-16 ins.; in other words, the normal motion of pin P is about 6 ins. The remaining 6 ins. will allow for any inaccuracies of adjustment, wear of pins in the movable eye-bars, etc.

The time required to operate the 256-ft. bridge illustrated, by engines or motors of varying HP. is shown in the following table:

	Time, secs.			Allowance for friction, wind, etc	
	For raising ends.	For opening bridge.	Total.	Raising ends.	Turning bridge.
6-HP.	30	114	144	50%	100%
10-HP.....	18	87	105	50%	84%
14-HP.....	13	73½	86½	50%	67%
18-HP.....	10	66	76	50%	51½%
22-HP.....	8¼	60	68¼	50%	50%

In this table the time for opening bridge includes that necessary to turn same through one quadrant, and was calculated by the methods of Johnson's "Framed Structures," adding to the theoretical HP. found necessary to overcome inertia and friction of rollers against turning, for wind, etc., the percentages shown; the time for raising ends is that necessary to raise the pin P through the 6 ins. found above, assuming the theoretical HP. to be increased 50% for friction and assuming the motor running at full speed under full load and at double speed under half-load or less.

It will be noticed from the above that the proportion of the total time which is used in raising the ends, especially where that total is small, compares very favorably with any ordinary end lift. The advantages claimed for the present design are, first, those due to its discontinuity; (1) All the stresses in the bridge are determinate; (2) the structure is more rigid under live loads than present constructions; (3) the load on the drum when the bridge is closed is decreased in the ratio of about five to four, and the live stresses in truss members throughout the middle half of the bridge are less; (4) there is no possibility of the ends hammering either at first or later. Secondly, the advantages of the mechanism shown over present styles of end lifts may be summarized thus: (1) The machinery is all concentrated at one point, directly under the eye of the operator for inspection and oiling, and is less liable to get out of order from being thus concentrated, and because all parts are fixed in position relatively to each other; the machinery is self-contained, whereas in the latter type the end piers form one of the working parts; (2) no rail lifts are required, with the attendant complications of loose rail-ends; (3) when the bridge is closed, no part of the lifting mechanism is under load, and the structure is in its most stable condition. As a final consideration, it may be said that the total cost

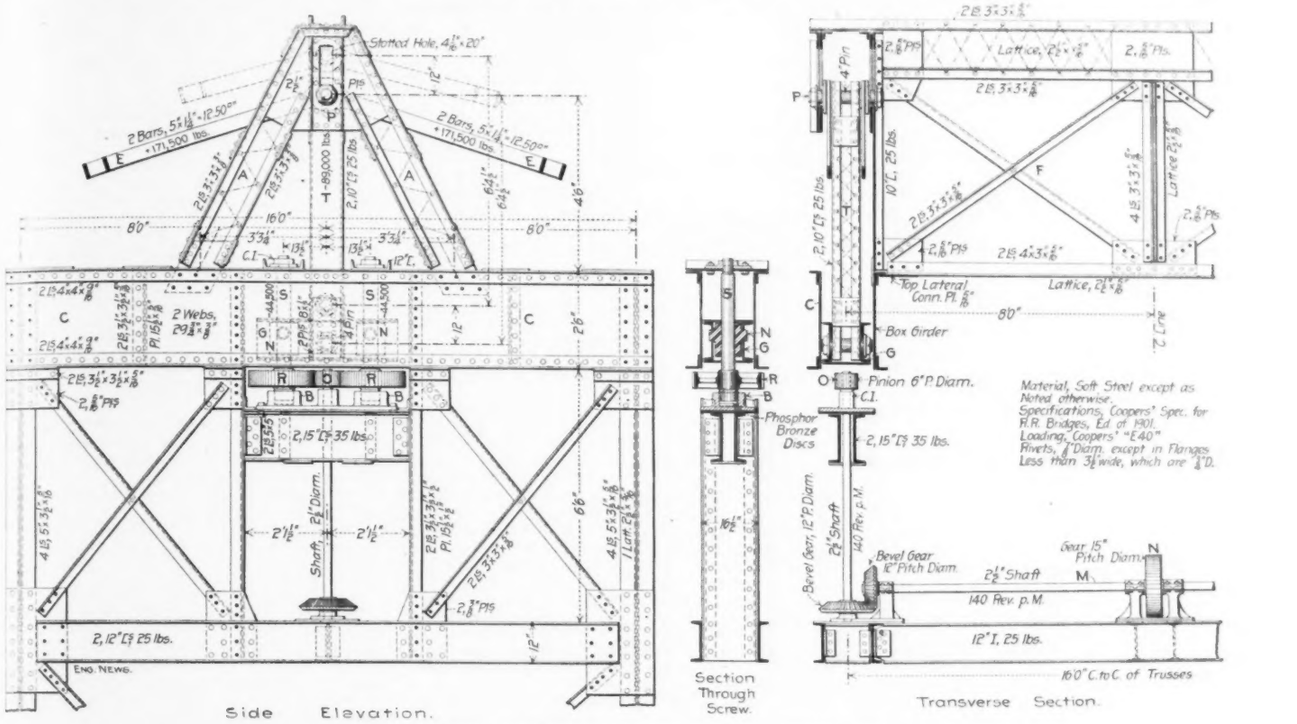

Side Elevation.
Section Through Screw.
Transverse Section.
Material, Soft Steel except as Noted otherwise.
Specifications, Coopers' Spec. for R.R. Bridges, Ed of 1901.
Loading, Coopers' "E40"
Rivets, ⅞ Diam. except in Flanges Less than 3½ wide, which are ⅞ D.
Pinion 6" P. Diam.
Phosphor Bronze Discs
Bevel Gear 12" P. Diam.
Bevel Gear 12" Pitch Diam.
Gear 15" Pitch Diam.
2½" Shaft
140 Rev. p.M.
Box Girder
Top Lateral Conn. Pl. ⅝"
Lattice, 2½"×⅝"
12"I, 25 lbs.
160" C. to C. of Trusses
2 Webs, 29⅜"×⅜"
2,15"Cl. 35 lbs.
2,12"Cl. 25 lbs.
Shaft 2½" Diam.
Eng. News.

FIG. 3. DETAILS OF TOP OF TOWER, 256-FT. DRAW SPAN.

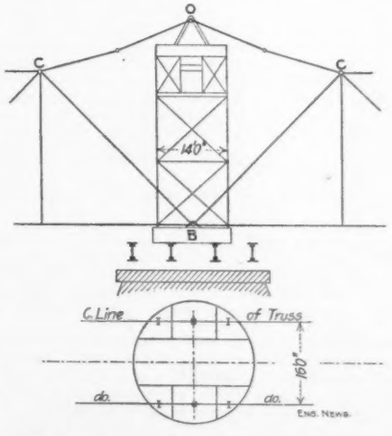


Fig. 4. Alternate Design for Center Section.

of the present design would probably differ little, in any case, and for spans of ordinary length, such as that illustrated above, would be less than that of a swing bridge constructed on the usual lines.

In connection with the design of this bridge the inventor offers an alternate arrangement of the central section that possesses interesting features. The object is to distribute all vertical loads uniformly over the bearing surface of the drum, instead of allowing unsymmetrical loading as with the constructions at present in use. Fig. 4 shows the proposed design as applied to a drum of greater diameter than the width of the bridge. As will be seen, a common center shoe is used for the two halves of each truss, the shoe being located on the center line of the span; a system of girders distributes the load from this pin to a number of points on the circumference of the drum. The tower load, which exists only when bridge is swinging, is also distributed uniformly by these girders, and it will be noticed that this tower takes the place of the "steadying wheels" used in center-bearing turntables, to make the structure stable against the overturning action of wind, etc., while swinging.

This scheme would add to the cost by increasing somewhat the length of each arm, but would ensure for a single-track bridge, and for double-track bridges under full load, a perfectly uniform loading on wheels and a centrally-applied load on the pier at all times. This would seem to be particularly adapted to bridges of moderate length.

THEORY OF THE STRENGTH OF BEAMS OF REINFORCED CONCRETE.*

By W. Kendrick Hatt.†

In a cast-iron beam, equal safety of both flanges is brought about by proper design of the form of cross-section. The center of gravity of section is brought near the tension flange. The outer fiber in tension is thus nearer the neutral axis than is the outer fiber in compression, and takes less stress. Concrete or cement mortar is also weaker in tension than in compression (ratio from 1 to 7 up to 1 to 10), but in concrete beams the change of form of cross-section in order to effect equal strength of flanges is impracticable or at least is not resorted to. The part in tension is usually strengthened by the insertion, near the outer face, of steel wires or other structural shapes. In the case of a beam (like an arch-ring) in which either face may be in tension, these steel forms are placed in both flanges.

The problem in mechanics is to analyze this case of flexure in order to derive an expression for the moment of resistance of such a beam; proceeding mainly from the ordinary assumptions of the theory of flexure. In case of concrete beams, however, we must modify these assumptions, for the value of E (the modulus of elasticity) in tension is not equal to that in compression. We must also assume that the elastic limit is passed in the concrete; for the concrete, being a porous material, is perfectly elastic only under very slight loads. It may be said that there is no widely accepted method of computation for the strength of concrete reinforced with steel; furthermore, the elements of the strength of the constituent parts are not well ascertained. The truth of the analysis must then be tested by further experiment.

THE STRENGTH OF THE CONSTITUENTS.

(A) Steel: This needs no description. The value of E is 30,000,000 lbs. per sq. in. The elastic limit may be taken as 40,000 to 50,000 lbs. per sq. in.

(B) Concrete (not reinforced): In compression the stress-strain diagram may be derived from a number of tests made in the Watertown Arsenal on concrete made up by workmen under conditions of service. (See Report of Tests of Metals, 1899, pp. 744-747.) The stress-strain diagram is a curve and the value of E depends on the part of the curve at which it is observed. The following values of E were obtained in the Watertown Arsenal tests on 12-in. cubes of Portland cement, broken stone concrete, mixed dry. These values are in pounds per sq. in:

Saylor's cement.	Age		
Concrete, 1 to 2 to 4:	Nine days.	Three months.	Six months.
Value of E.....	1,657,000	3,461,000	4,500,000
Load	1,000	1,000	1,000
Crushing strength ..	1,944	2,200	3,500
Concrete, 1 to 3 to 6:			
Value of E.....	1,956,000	3,750,000	2,812,000
Load	1,000	1,000	1,000
Crushing strength ..	2,308	2,500	3,500

In these tests the load was released at regular intervals in order to obtain the set. The values of E given, which are quoted from a large number of results, are computed from the compression obtained by subtracting the previous set from the compression due to the load of 1,000 lbs. per sq. in. Smaller values would result if the compression were reckoned from the primitive length of block. The method followed seems proper for the case in hand.

The modulus of elasticity of gravel concrete is greater than that of stone concrete, while that of cinder concrete is less. It is thought by the writer that the Watertown values are proper for use, at the stresses for which they are given, in the case of broken stone "dry" concrete. An examination of stress-strain diagrams plotted from these results shows that a parabola closely approximates the true curves. The area will be ⅔, and the moment ⅝ that of the rectangle surrounding the parabola. The value of E varies with the amount of water used in mixing. Wet mixtures have a modulus much lower than that of plastic mixtures. The modulus of elasticity is a very important element in the calculation of the strength of reinforced concrete beams, and should be obtained for the actual material used in each case.

Fig. 1. Stress-Strain Diagram for Reinforced Concrete Under Tension.

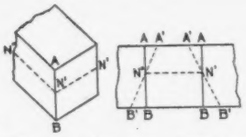


Fig. 3. Diagram Illustrating Position of Neutral Axis.

(C) Reinforced Concrete under Tension: Observations of the deformations of reinforced concrete under load are rare. M. Considère, a noted French investigator, says that concrete and mortar when not reinforced break with an elongation of less than 1 part in 10,000. Yet, he remarks, the tension faces of a reinforced concrete beam support an elongation of more than 1 in 1,000 without sign of fissure, or 10 times the former amount. (This 1 in 1,000 is nearly the elongation of steel at its elastic limits.) He remarks ("Memoires de l'Academie des Sciences," Dec. 12, 1898), that the tension modulus of elasticity of reinforced concrete remains sensibly constant up to a stress which corresponds to the rupture in case of ordinary concrete not reinforced. When the deformation of the reinforced concrete surpasses the latter limit, E becomes practically zero. The stress-strain diagram for reinforced concrete is as shown in Fig. 1.

M. Considère loaded one beam of reinforced concrete, until the outer fiber of the tension flange was under an elongation of 1 in 500; and afterwards subjected the beam to 140,000 repetitions of a load which caused an elongation of 1 in 2,000 to 1.27 in 1,000. He then detached from the tension flange of this beam a small bar of mortar; on testing this he found no decrease of strength due to the service. The point to notice is that we can count on very large deformations in the material of the tension flange. The stress in the steel reinforcing material may be near its elastic limit, and yet the concrete in tension may still be carrying load without cracking. This element of strength may then be included in the analysis.

The excess of elongation over the ordinary concrete may be accounted for by supposing that the wire or other material reinforcement distributes an elongation throughout the entire length of the concrete; whereas the elongation of the concrete when not reinforced is confined to the section of rupture. To accomplish this strengthening, however, the iron or steel reinforcement must be of sufficient area and near enough to the tension flange. The prevention of cracks is, however, a feature of more importance than the increase of strength mentioned. We may assume that the ratio of tensional to compressive strength of concrete is the same as that occurring in case of mortar, namely $^1/_{10}$. The tensional strength of the concrete will then be $^1/_{10}$ of the values given in the preceding table, or 220 lbs. per sq. in. for a mixture of 1 to 2 to 4 at three months. This value may be checked by some tests by the writer on bars of concrete broken at three months. They were of 1 to 2 to 4 mixture, 6 × 6 ins. cross-section, and 30 ins. span. They broke under an average load of 2,300 lbs. Now calculating the strength of these concrete beams in order to obtain the fiber stress in tension, not by the ordinary assumption of elastic deformation, but by taking account of the true nature of the stress-strain diagrams, the tensile strength of the concrete in these beams is found to be 210 lbs. per sq. in.

THE COMPUTATION OF THE STRENGTH OF A CONCRETE BEAM.

The problem of computing the strength of a reinforced concrete beam is not difficult when once the basis of computation is fixed. Various solutions are obtained under various assumptions as to the position of the neutral axis, and the share which the material of the tension flange has in the resistance of the beam. The case will be worked out below in the more general method, and the various approximations introduced later. The writer follows M. Considère,* with some modifications, in the analysis below.

A material like concrete or mortar will not be perfectly elastic even for very light loads.‡ The stress-strain curve is curved from the start. We therefore cannot assume that Hooke's law holds; that is to say, we cannot assume that the stress increases directly as the strain. We must base the analysis on the actual stress-strain curve. The problem will be to compute the ultimate or breaking load of the beam, and not, as is the case in the formula for steel beams, the elastic strength.

ASSUMPTIONS: In deriving a formula for the moment of resistance of the beam, we assume:

1. The cross-sections of the beam remain plane surfaces during flexure. Looking at Fig. 3, this means that two sections originally parallel, will, after the imposition of the load rotate to A' B'. (This has been shown to be the case experimentally. See "Trans. Am. Soc. C. E.," July, 1901.) The material or "fibers" of the upper flange will be under compression; the fibers of the lower flange will be under tension. Somewhere between the two will be a surface of material under no extension. This surface is called the neutral surface. Its intersection N N' with the cross-section of the beam is called the neutral axis. The location of this axis is part of the computation below.

2. The applied forces are perpendicular to the neutral surface.

3. It is assumed that the presence of the material surrounding any elementary fiber will not modify the effect gate or be compressed just as if it were under that load by itself in a testing machine.

4. There is no slipping between the faces of the wire and the surrounding concrete.

5. The elastic limit of the concrete is exceeded in both the compression and tension flange.

6. There are no initial stresses due to shrinkage or expansion of the concrete while setting.

VARIATION OF STRESSES OVER CROSS-SECTION: Starting with these assumptions, we may proceed to determine the variation of stress over the cross-section for any given deformation of the beam. If for instance, we consider the upper face in compression, the law of increase of stress as we go from the neutral axis out toward the upper surface must be represented by the stress-strain diagram of an ordinary compression test. For the rate of increase of deformation of the fibers in the beam is directly as the distance from the neutral axis; and this increase of stress for regularly increasing deformation is what is shown by the stress-strain diagram. If then we apply the stress-strain diagram for compression directly to the cross-section of the beam as is shown in Fig. 4, we will have a representation of the stress over the cross-section above the neutral axis thereof.

In like manner we obtain the representation of the increase of stress over the tension face, as far as the concrete is concerned, by applying thereto the stress-strain diagram for reinforced concrete tested in tension. Finally the stress due to the steel reinforcement may be represented by a force acting at the location of the wire or other shape of steel. The entire state of stress is shown in Fig. 4.

Before we can compute the strength of this section we must assume the following quantities: Moduli of elasticity of steel and concrete; tensile and compressive strength of concrete; sizes of beam and reinforcement with location of latter. The distance of the steel from the lower face is supposed known; the tension in the concrete on the tension face is supposed equal to the ultimate strength of the reinforced concrete in tension, say 200 lbs. per sq. in. This last assumption is justified by the remarks above concerning the large extensions experienced by reinforced concrete under tension.

There are three unknown quantities to be determined: (1) The distance of the neutral axis from the upper face, (2) the stress in the steel, and (3) the moment of resistance of the beam. These three unknown quantities are

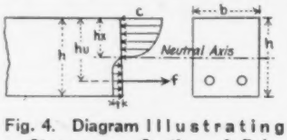


Fig. 4. Diagram Illustrating Stresses on Section of Reinforced Concrete Beam.

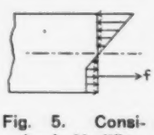


Fig. 5. Considere's Modification of Fig. 4.

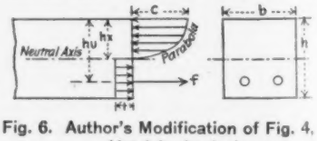


Fig. 6. Author's Modification of Fig. 4, Used in Analysis.

*Abstract of a paper presented at the annual convention of the Indiana Engineering Society, at Indianapolis, Ind., Jan. 29, 1902.

†Professor of Applied Mechanics, Purdue University, Lafayette, Ind.

*"Methode d'Epreuve des Constructions en Béton Armé." Congrés International des Methodes d'Essai. Paris, 1900. Vol. II., p. 330.

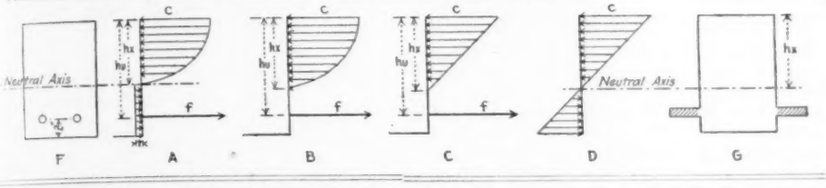
‡The stress at which permanent set was obtained in Watertown tests varies from 100 to 500 lbs. per sq. in.

Results of Various Theories for Computing Strength of 8 × 12-in. Reinforced Concrete Beams; 10-ft. Span.
E_c = 2,000,000; E_s = 30,000,000; 2 rods 2 ins. from lower flange.

	400(b) 200(a) 2,800				400(b) 200(a) 2,800				500 3,600				500 3,600			
p	1/80 = two ⅞-in. rods.				1/40 = two 1¼-in. rods.				1/80				1/40			
Theory	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
X ratio	(a)0.388 (b)0.414	0.363	0.402	.....	(a)0.476 (b)0.494	0.335* 0.557 0.456	0.500	.....	0.411	0.363	0.402	.....	0.493	0.456	0.500	.....
h x, ins.	4.97	4.36	4.82	6.63	(b)5.93	4.02 6.58 5.47	6.0	7.04	4.93	4.36	4.82	6.63	5.92	5.47	6.0	7.04
f lbs. per sq. in.	(a)48,200 (b)42,500	54,170	45,020	.....	(a)31,500 (b)28,800	25,000 41,600 34,692	28,000	....	55,400	69,700	57,900	....	37,240	44,500	30,000	.....
M, in.-lbs.	(a)450,000 (b)563,000	543,700	437,000	609,000	(a)658,000 (b)667,000	294,000 747,000 655,500	550,000	132,000 665,000	728,000	700,000	581,000	134,000 782,000	857,000	840,000	691,200	168,000 854,000
Center load, lbs.	(a)15,000 (b)18,766	18,120	14,556	20,300	(a)21,900 (b)22,248	9,830 24,900 21,852	18,300	4,400 22,200	24,260	23,300	19,400	4,400 26,060	28,560	28,032	23,000	5,600 28,400

*Values in column B are for three values of Ec: 5,000,000, 1,000,000, 2,000,000.

The greater values in column D are for strength of compression fiber; the smaller are for strength of tension fiber.



determined by the statement (in algebraic language) of three facts, namely, (1) The total force of compression on the compression side equals the total force of tension (i. e., of steel plus concrete) on the tension side; (2) the extension of the steel and the compression of the concrete at the upper fiber will be to each other as the distance of these materials from the neutral axis; (3) the moment of the external forces on one side of the section about the neutral axis of the section equals the moment of the resisting stresses on the section about the same axis.

Since the stress-strain curves are curves without any simple algebraic equation, some assumption as to their nature must be made. M. Considère considers that the case may be represented very nearly by a straight line drawn as shown by Fig. 5. Other writers neglect the tensional strength of the concrete altogether; while still others suppose the concrete to be wholly elastic, and thus draw two straight lines for the expression of the relation between the stress and strain in place of the two curves shown in Fig. 4. Those methods will be dealt with later.

In the analysis below the writer will assume the curve of variation of compression stress to be a parabola, drawn with its vertex at the point of highest strength. The justification for this procedure is found by an examination of the values given in the Watertown report above referred to. Plotting these stress-strain diagrams and computing the area and moment of each about the axis of loads, the average values for 12 cases show 0.657 and 0.453 where the theoretical values are 0.666 and 0.417 respectively. It appears that the assumption of a parabola is a close approximation to the original curve.

The tensional stress diagram will be assumed to be a straight line parallel to the section. This involves a slight error on the side of danger, counting about 5% more area adjacent to the neutral axis than exists in the tensional stress area of the tension side. The effect of this on the moment may be disregarded.

ANALYSIS.

Proceeding to the algebraic work, and referring to Fig. 6, in which h and b are respectively the depth and width of the beam (supposed rectangular).

h x = the distance to the neutral axis from the face of the compression side;
h u = the distance to the reinforcement (only one side considered reinforced);
F = area of concrete;
F′ = area of steel;

$$p = \frac{F'}{F} \text{ (usually from } 1/75 \text{ to } 1/100).$$

Here x, u and p are ratios.

E_c = modulus of elasticity of concrete;
E_s = modulus of elasticity of steel;
f = stress on steel reinforcement;
c = compressive stress in outer fiber of concrete;
t = tensional stress in outer fiber of concrete.

Referring to Fig. 6, we may derive the following relations:

Area in compression diagram = ⅔ c h x;
Total force on compression flange
= ⅔ c h x b;
Moment of compression about neutral axis
= ⅔ c h x b ⅝ h x;
(This ⅝ h x is the distance of the center of gravity of the parabola from the neutral axis);
Total force of tension on concrete
= t h (1 − x) b;
Moment of this tension about neutral axis

$$= t\,h\,(1-x)\,b\,h\,\frac{(1-x)}{2};$$

Tension on steel = F′ f = p h b f;
Moment of this tension on steel about neutral axis
= p h b f h (u − x).

Now, since the total tension acting to the right must equal the total compression acting to the left, we have:

$$\tfrac{2}{3}\,c\,h\,x\,b = t\,h\,(1-x)\,b + p\,h\,b\,f; \text{ or}$$
$$\tfrac{2}{3}\,c\,x = t\,(1-x) + p\,f. \qquad (1)$$

Then taking moments about the neutral axis, the moment of resistance of the section, about the neutral axis, M is:

$$M = \tfrac{2}{3}\,c\,h\,x\,b\,\tfrac{5}{8}\,h\,x + t\,h\,(1-x)\,b\,h\left[\frac{1-x}{2}\right] + p\,b\,h\,f\,h\,(u-x);$$
$$= b\,h^2\left[t\,\frac{(1-x)^2}{2} + \frac{5}{12}\,c\,x^2 + p\,f\,(u-x)\right]. \qquad (2)$$

Next we may state the relation of f and c, the stress in the steel and concrete respectively.

By definition of the modulus of elasticity $E_c = \frac{c}{e_c}$ where e_c is the unit compression experienced by the concrete on the upper face of the beam. Likewise $E_s = \frac{f}{e_s}$.

But, since the sections of the beam were assumed to remain plane surfaces

$$\frac{e_c}{e_s} = \frac{h\,x}{h\,(u-x)} \text{ or, } \frac{c}{f} = \frac{E_c}{E_s}\,\frac{x}{(u-x)}. \qquad (3)$$

If we eliminate f from (1) and (3):*

$$t\,(1-x) + p\,c\,\frac{E_s}{E_c}\,\frac{u-x}{x} = \tfrac{2}{3}\,c\,x. \qquad (4)$$

These are the equations needed to compute the strength of a reinforced concrete beam. From (4) we may first obtain the distance to the neutral axis (after having assumed the tensile and compressional strength and modulus of elasticity of the concrete, the location and number of the reinforced shapes with the modulus of elasticity of the steel).

Having found x, the value of f is computed from (3), and finally the resisting moment of the section is found from (2).

OTHER METHODS OF COMPUTATION.

If we neglect the tensional strength of the concrete we will have a state of stress as shown at B in the diagram accompanying the table.

The equations then become:

$$\tfrac{2}{3}\,c\,x = p\,f. \qquad (1')$$
$$M = b\,h^2\,[\tfrac{5}{12}\,c\,x^2 + p\,f\,(u-x)]. \qquad (2')$$
$$p\,\frac{E_s}{E_c}\,(u-x) = \tfrac{2}{3}\,x^2. \qquad (4')$$

If we neglect the tensional strength of the concrete and consider the compression diagram to be a straight line, the state of stress will be given by C in the diagram.

The equations become:

$$\tfrac{1}{2}\,c\,x = p\,f; \qquad (1'')$$
$$M = b\,h^2\,[\tfrac{1}{3}\,c\,x^2 + p\,f\,(u-x)]; \qquad (2'')$$
$$\tfrac{1}{2}\,x = p\,\frac{E_s}{E_c}\,\frac{u-x}{x}. \qquad (3'')$$

Or, finally, we may use the method outlined by Professor Johnson in Engineering News; that is, we may suppose the concrete elastic until rupture, and an equivalent beam to be formed by replacing the area of the steel by an area of concrete equal to the area of the steel multiplied by the ratio of the moduli of elasticity of steel and concrete respectively. Thus a beam would be modified from F to G, where the shaded area is equal to the area of the steel multiplied by the ratio of E_s to E_c, that is about 15 times the area of the steel.

To compute the strength of this modified or equivalent beam we have to find its neutral axis (which passes through the center of gravity); and compute the moment of inertia about this neutral axis. This case will be found worked up in Engineering News Jan. 3, 1895, by Prof. J. B. Johnson.

To compare the results of these methods applied to beams under various assumptions as to strength of material the writer has computed the accompanying table, to which the proper diagrams are added. It will be noticed that a concrete with a lower value of the modulus gives a stronger beam than one of higher modulus—other things being equal.

DOUBLE REINFORCEMENT.—This case may be easily worked out on the same lines as the preceding.

OTHER ELEMENTS AFFECTING STRENGTH OF REINFORCED CONCRETE BEAMS.—The question of initial strains due to shrinkage of concrete while setting in air or its expansion while setting in water; or, the question of stresses due to the difference of coefficient of expansion of the concrete and steel on change of temperature; or finally, the slipping of the concrete on the reinforcement during service either from shock or steady stress are not considered in this paper.

The effect of the initial strain set up by expansion or contraction while hardening in water or air respectively modifies the deformations experienced under a load, so that a beam set in air will deflect to a greater extent under a given load than a beam set in water. M. Considère says that this shrinkage or expansion may produce an actual sliding of the concrete on the steel, which may be prevented by transverse reinforcements. Considering all these elements and the variation of the value of E in wet or dry mixtures, it is evident that the use of refined theorises of computation is not justified.

The theory marked B seems to the writer to be simple and on the side of safety.

The application of these theories to problems of design may be rendered more easy by properly prepared graphical diagrams from which values may be read. The experimental side of the problem is not well worked out. Experiments are needed to determine the tensile strength of the concrete, reinforced and not reinforced; and a long series of experiments to check up the reliability of the formulas used in design.

*The parabola as assumed supposes E_c to be zero at the compression face. E_c in computation must be obtained at a point nearer the neutral axis.

BOOK REVIEWS.

DIFFERENTIAL AND INTEGRAL CALCULUS.—For the use of colleges, universities and technical schools. By E. W. Nichols, Professor of Mathematics in the Virginia Military Institute. Cloth; 6 × 8 ins.; pp. 394; 74 illustrations. D. C. Heath & Co., Boston, Mass. Price, $2.00.

This work is quite similar to other modern college textbooks on calculus; a special effort appears to have been made, however, to present concrete examples at all possible points. A chapter on "Mechanical Applications," introduced in this connection at the close of the book, will help to bridge the gap, so wide to most studpnts, between calculus and its applications in the subjects of mechanics and physics. On this account also a chapter on "Differential Equations" forms a valuable feature of the book. The omission of a summarized table of integrals will probably be felt with regret by many who might desire to use the book as a reference.

SCHOOL HYGIENE.—By Edward R. Shaw, Professor of the Institute of Pedagogy, New York University. New York: The Macmillan Co. Cloth; 5 × 8 ins.; pp. 260; 59 illustrations.

While treating the subject in hand from the standpoint of an educator, as is fitting for a book in a series known as the "Teachers' Professional Library," this volume accords with the principles of sanitary science and common sense. After discussing school buildings and grounds, warming and ventilation, sanitation, and the comparatively new yet important subject of school baths, the author takes up the no less important but perhaps even more neglected subjects of school furniture and its relations to the health of the pupil, handwriting and like topics. The book is well illustrated, and has a bibliography and an index.

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Why is the bill permitting Massachusetts cities to build and operate subway conduits for electric wires defeated year after year in the State legislature? The only reason we can see is that the lobby, in the interests of the companies who might be compelled to place their wires in such conduits, and pay the cities rental therefor, is strong enough to throttle the measure. The present is the sixth year, according to the Boston "Journal," in which such a bill has been introduced in the legislature. Last year twelve cities asked for its passage, but, as was the case the year before, the Senate killed it after the house had passed it. The bill appears to safeguard the interests of taxpayers and private corporations alike, and its passage would undoubtedly prove of mutual benefit to the cities and companies of the State if only the short-sighted opposition of the latter could be overcome.

The cost of inadequate fire protection is partly shown by a notice sent out by insurance companies to property-owners in the down-town section of Pittsburg. The notice states that:

> The annual rate on this policy is increased 20 cts. per $100 until a system of water mains and fire hydrants satisfactory to this company shall be provided by the City of Pittsburg.

If the city would take effective steps to cut down its enormous water waste, as recommended by its filtration commission and engineers in 1899, the problem of providing adequate fire protection would at least be lessened.

Two notable achievements in the struggle against water waste, and also in behalf of a fair and equitable basis for the sale of water, mark the beginning of the present year. These are the award of a contract at Cleveland, O., for about 11,000 water meters, and at Columbus, O., for about 10,000. These contracts doubtless mark the beginning of metering on a large scale which will continue with increasing rapidity as time goes on.

Another illustration of the dangers from fire in electric cars equipped in the ordinary manner was furnished, in Brooklyn last week, on a crowded Greenpoint car of the Brooklyn Rapid Transit Co. The "Tribune" gives an account of the accident as follows:

> The controller box exploded with a loud report and a vivid accompaniment of blue and red flames. William Weick, the motorman, leaped over the gate of the platform into a snowbank. Some one in the car cried "Fire!" and a wild rush for the door ensued. Some were injured in the crush, others were badly bruised in falling in the icy street, and others suffered from nervous shock. These persons were treated by a surgeon from the Eastern District Hospital, and were taken to their homes.

If such panic occurs in a street car from which escape is easy, what would be the result of such an accident on a car or train on an elevated railway or in a long tunnel? The point to be emphasized, of course, is that greater precautions should be taken in the wiring and the installation of electrical apparatus on cars. In fact, this is already being done. In the equipment of the New York Rapid Transit Ry., for example, we are informed that the greatest precautions are to be taken to avoid accidents from the electrical equipment, and to make all parts which can possibly be reached by fire from an electrical burnout of incombustible material.

The very unsatisfactory character of the lighting in a large proportion of the electric street cars in service is commented upon in a recent issue of the New York "Times," and it might well have made its criticisms even more severe. Take it all in all, the average steam railway passenger coach since the general introduction of Pintsch gas is much better lit than the average car propelled by electricity. The constant variation in voltage on street railways, and the large drop in voltage on outlying portions of the system, combined with the occasional flickering or entire extinguishing of the lights, combine to make the system little short of a public nuisance. The electric conduit system is a considerably worse sinner in this respect than the overhead trolley, for at curves and switches the plow is continually making and breaking contact. The system of taking current directly from the trolley circuit for lighting the car has the merit of cheapness and simplicity, but the injury to the eyes of passengers who attempt to read by a constantly varying light, is such that the public is entitled to demand something better. It ought to be possible to equip cars with storage batteries to steady the lights and supply the deficiencies in the trolley current; or if this is not feasible, the adoption of gas illumination might be worth while. Indeed, we believe gas lights have actually been used on a few electric roads.

The ordinance for the prevention of electrolysis of water mains at Norfolk, Va., noted elsewhere in this issue, very correctly puts the whole burden of determining and applying means of prevention upon the electrical companies whose currents are involved. Another good feature of the ordinance is its provision for yearly and, if the city deems best, special tests to determine whether or not danger of electrolysis exists.

The seventieth anniversary of the founding of the Baldwin Locomotive Works, which is celebrated at Philadelphia to-day, is an event deserving more than passing notice. Almost coincident with the anniversary is the completion by the works of its twenty-thousandth locomotive. Of course, the early history of the works was in the days of small things. Less than a thousand locomotives were built by the works from the time Mr. Baldwin built his first locomotive, "the Ironsides," in 1832, until his death in 1866. Of the 20,000 locomotives which have been built at the works, half have been built during the past thirteen years. Even these figures do not represent fully the contrast between the business of early years and that of the present time on account of the great increase in the average weight and power of locomotives during the past dozen years.

If we take the average price received by the works for its entire output at $11,000 per locomotive, it appears that the entire product of the works since its establishment reaches the huge value of $220,000,000. This, it is to be noted, is the work of a single manufactory, which, since its establishment, has been managed by its actual owners, who have been associated in a partnership and not as a corporation. The success achieved has been attained through the sound and conservative management and the wise enterprise of those who have had the responsible charge, and not through any of the artifices of modern financiering whose praises have been so widely sung in recent years.

CONCERNING PERMISSION TO A CONTRACTOR TO DEPART FROM SPECIFICATIONS.

An unfortunate controversy has arisen in New Orleans in connection with the work on a system of drainage for the city. As many of our readers will doubtless remember, the plans for this work were prepared a half-dozen years or so ago by a Commission consisting of Messrs. H. B. Richardson and B. M. Harrod of New Orleans, and Rudolph Hering, of New York city, acting in conjunction with Mr. Linus W. Brown, then City Engineer of New Orleans. All those named are Members of the American Society of Civil Engineers. In 1897, construction was begun under a special Commission, with Mr. B. M. Harrod as Chief Engineer, and has been in progress ever since.

For some time the New Orleans newspapers have been teeming with charges that Mr. Harrod had allowed certain contractors to use cheaper cement in their work than was called for by the specifications, and at a recent meeting of the Commission a report upon the matter was presented by a special committee of the Commission and also a statement by Mr. Harrod himself. We print Mr. Harrod's statement practically in full as follows:

> The general specifications adopted by the city council in 1896 were vague concerning the quality of cement to be used, the properties required and the tests to be exacted. In the case of imported Portland cement, it was only required that 95% should pass through a 2,500-mesh sieve and that a seven-day heat test should give a tensile strength of 400 lbs. per sq. in.
>
> The special specifications for the contracts of 1897 with the National Contracting Co., and of 1899 with C. F. Collom & Co., added nothing to these requirements beyond calling for bids on the following items:
>
> Contract "A"—National Contracting Co.: Central power and pumping stations and lined and covered canals. Brickwork—American Natural cement (1 of cement, 2 of sand); imported Portland cement (1 of cement, 2 of sand); imported Portland cement (1 of cement, 3 of sand). Concrete—American Natural cement (1 of cement, 2 of sand); imported Portland cement (1 of cement, 2 of sand); imported Portland cement (1 of cement, 3 of sand).
>
> Contract "F"—C. F. Collom & Co.: Lined and covered canals. Brickwork—American Natural cement (1 of cement, 2 of sand); Portland cement (1 of cement, 3 of sand). Concrete—American Natural cement (1 of cement, 2 of sand); imported Portland cement (1 of cement, 3 of sand). Surfacing—Portland cement (1 of cement, 1 of sand).
>
> These contracts afford the only instance where Steel Portland has been used in the place of imported Portland cement, and in each case it is left undetermined in the specifications, and presumably at the discretion of the engineer, in what part of the work the different material bid for should be used.
>
> Subsequent to the adjudication of these contracts, application was made by the contractors to be allowed to use Steel Portland cement, an American product, where the bid had been for imported Portland. Additional conditions, both physical and chemical, were then attached to the specifications, and the contractors were informed that if that cement complied with the higher or more detailed tests its use would be allowed. The additions to the specifications required much finer grinding, certain tensile strength at different proportions. The material used under this agreement was under constant test at Tulane University, and its acceptance or rejection was determined by these tests. It proved satisfactory under the tests, as was also the work in which it was used.
>
> The advantages which may have accrued to these contractors from the use of a cheaper cement than was specified may be estimated in the following manner. The measurements for final estimates for payment on the part

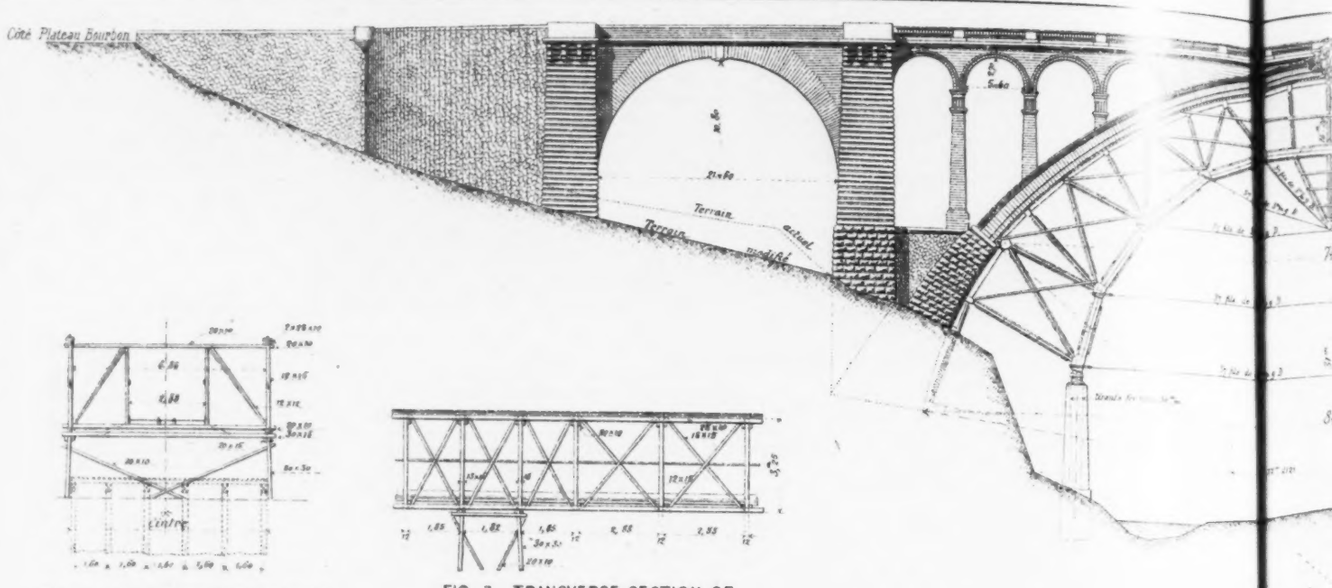


FIG. 8 ENLARGED ELEVATION OF SERVICE TRESTLE.

FIG. 7. TRANSVERSE SECTION OF SERVICE TRESTLE.

FIG. 1. ELEVATION O

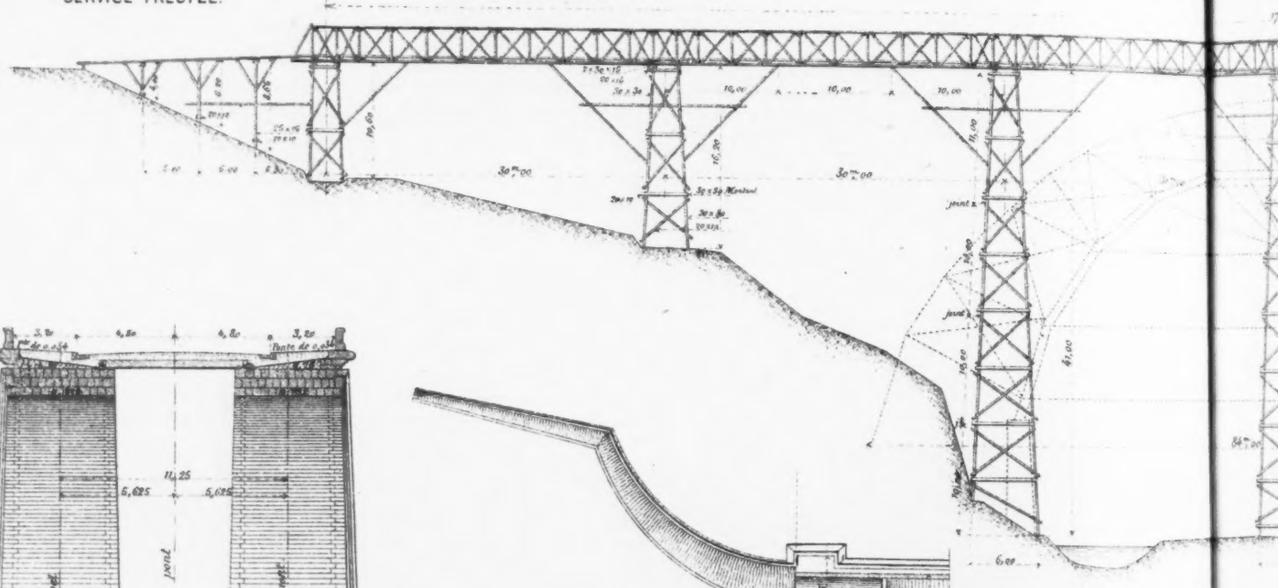


FIG. 6. ELEVATION OF SERVICE TR

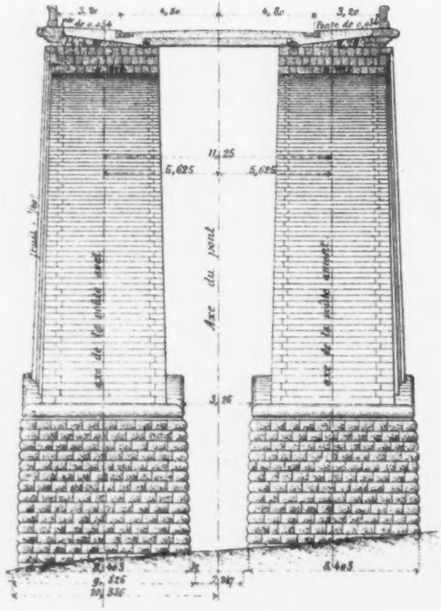


FIG. 4. TRANSVERSE SECTION AT CROWN OF ARCH.

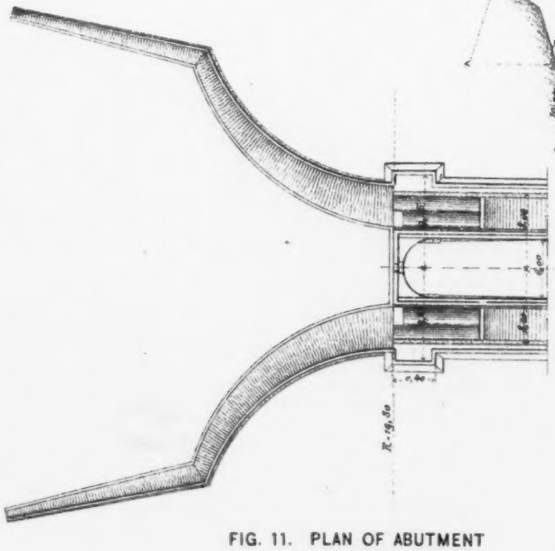


FIG. 11. PLAN OF ABUTMENT AND WING WALLS.

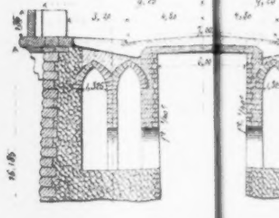


FIG 5. TRANSVERSE SECTI AT ABUTMENT PIER.

THE LUXEMBURG BRIDGE; THE LONGEST SPA

M. Sejourne, Chief Engineer.

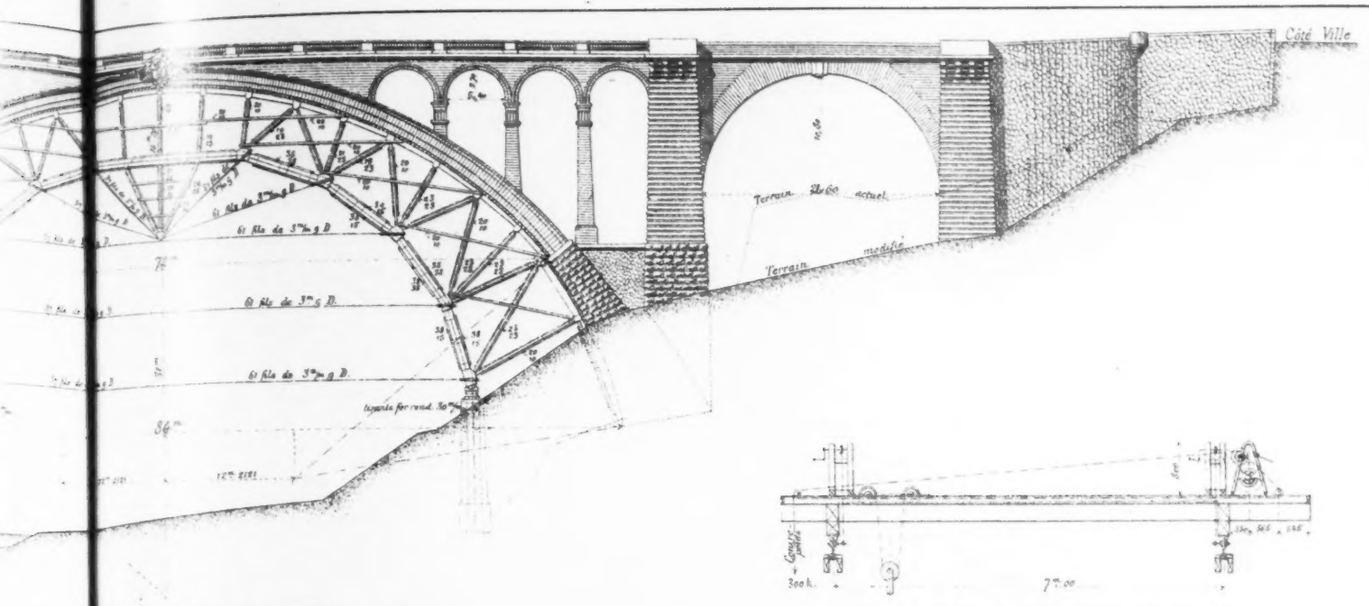
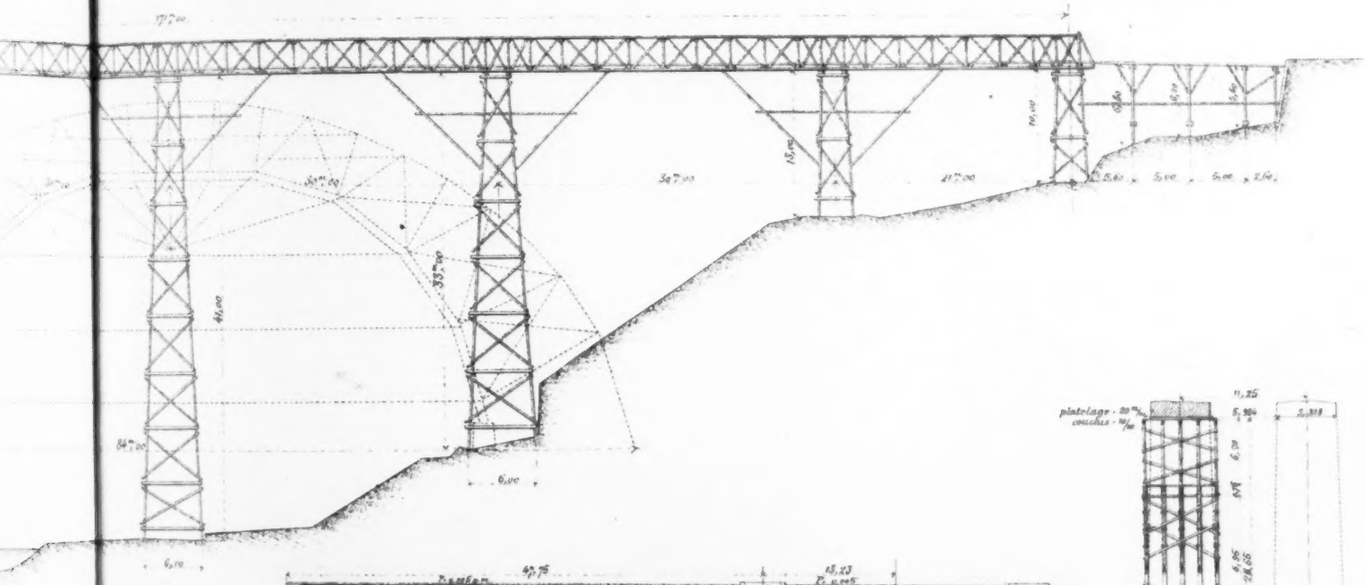
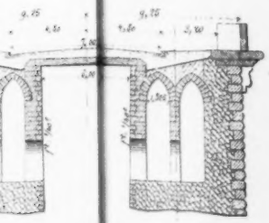


FIG. 1. ELEVATION OF ARCH AND CENTER.

FIG. 9. TRAVELER FOR HANDLING MATERIAL.



EVATION OF SERVICE TRESTLE.



5. TRANSVERSE SECTION AT ABUTMENT PIER.

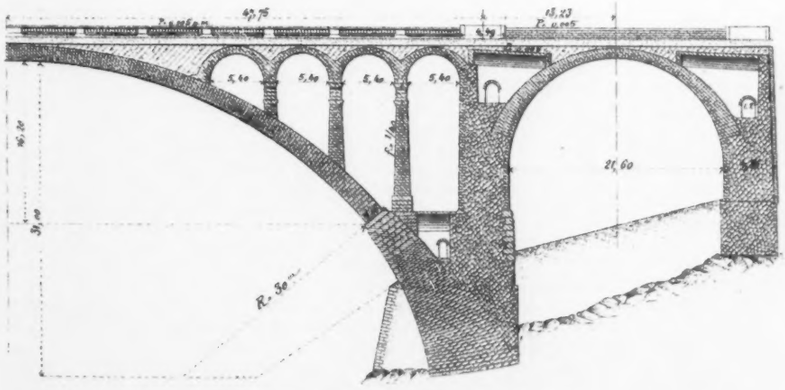


FIG. 3. HALF LONGITUDINAL SECTION.

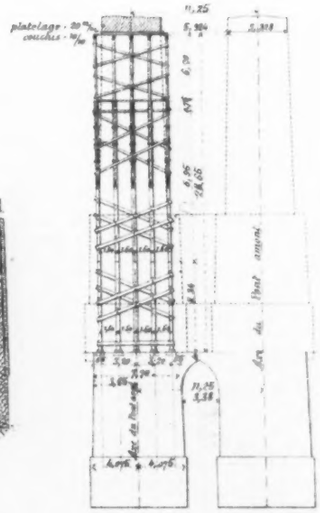


FIG. 10. TRANSVERSE SECTION OF ARCH CENTER.

E LONGEST SPAN MASONRY ARCH IN THE WORLD.

Fougerolle Bros., Paris, France, Contractors.

CHAS. HART, LITH., 88 VESEY ST., N. Y.

of these two contracts where Steel Portland instead of imported Portland cement was used were as follows:

	Brickwork, cu. yds.	Concrete, cu. yds.
National Contracting Co.....	46,099	33,109
C. F. Collom & Co.....	.....	4,842
Total	46,099	37,951

Under the contracts let in 1901, propositions were received from the contractors to use Steel Portland instead of the other grades of American Portland cement, at a reduction of 18 cts. per cu. yd. of brickwork, and also to use these other brands of American Portland instead of imported Portland cement, at a reduced price of 8 cts. per cu. yd. The Steel Portland could, therefore, be used in brickwork for 26 cts. less per cubic yard than the imported Portland cement. The difference in concrete would be about twice this amount, or 52 cts. per cu. yd. Applying these prices to the quantities given above gives a difference in favor of the contractor of $31,720.26. This is the difference as it existed last year, based on propositions of the contractors. I have no information whether it would have been more or less in 1897.

The price at which Steel Portland cement could be obtained, allowed the contractors a larger profit than there was in the use of imported Portland, but the tests under which it was used were more complete and exacting than those which were provided in the general and special specifications under which the bids were made, and the material proved satisfactory under these tests and in the work.

I recognize the justice of the censure conveyed in the report of your committee in regarding this transaction as an error, inasmuch as better terms might have been exacted from the contractor, and as it should have been referred to the Commission for its sanction.

The report of the special committee above referred to contains no facts additional to what are given above. The censure alluded to was merely to the effect that the engineer should not have permitted a departure from the specifications of such importance without obtaining the approval of the Commission.

It is only too evident from both the tone and the matter of the comments in the New Orleans newspapers, that personal jealousies, and very likely trade rivalries, are really at the bottom of the attacks on Mr. Harrod. We can, of course, give no place to a discussion of these. What we do wish to call particular attention to is the exceeding care which an engineer must take if he would avoid such difficulties as that into which Mr. Harrod has unfortunately fallen.

Impartially analyzed, Mr. Harrod's offence consists in this: that he permitted the contractor to depart from the strict letter of the specification and to use a material claimed to be of less cost than that specified, but believed to be equally good for the purpose. It is safe to say that if every engineer who has ever favored a contractor in this way were to be held up to public condemnation, a large proportion of the ablest and most honored members of the profession would have to suffer.

We strongly urge the public and the press of New Orleans not to be misled in this matter into unjust condemnation of their engineer. Mr. Harrod has received at the hands of his fellow-members in the civil engineering profession the highest honor in their gift—election to the Presidency of the American Society of Civil Engineers. Surely a man who has been thus rated by those best qualified to judge of his professional qualifications is entitled at least to fair and impartial treatment by the general public.

We shall not attempt to discuss the case in detail; but there are a few things which should be said. First: It was a mistake to specify "imported Portland cement" for the work, although it was a mistake which many conservative engineers were still making in 1896, when the specifications for the New Orleans drainage works were drafted.

Second: It is not only the right but the duty of the engineer in charge of construction to seek to carry out the spirit rather than the letter of specifications. Engineers frequently permit and even require contractors to depart from the letter of their specifications, and it is often to the interest of the engineer's employer, or, in other words, to the benefit of the work that this should be done. All the conditions which will arise while a piece of work is under construction cannot be foreseen by the engineer when he makes his specifications and draws his plans at the outset.

Taking the specific case of the New Orleans specifications, as we have said above, the insertion of "imported Portland cement" as a requirement in the specifications was probably due to conservatism on the part of the framer of the specifications; as we have said above, the inserit developed that equally good cements of American manufacture could be obtained, why should the letter of the specification be insisted upon?

The opponents of Major Harrod aver that the Steel Portland cement, which he permitted to be used, was an inferior material. This is true in so far that no one would claim it to be equal to the best Portland for all uses; yet for certain purposes it has been and is being largely used, we believe, by reputable engineers. No one is justified in passing an opinion on the wisdom and propriety of permitting its substitution without an expert knowledge of the situation where it was to be used.

The critics of Mr. Harrod, however, declare that even if the substitution of the Steel Portland cement for the Portland were permissible, the contractor should have been compelled to reduce his contract price by the amount which he saved by the use of the cheaper material. As Mr. Harrod failed to do this they hold him up to severest censure, and apparently consider that his betrayal of trust is proved beyond the possibility of a doubt.

On this point again we would counsel the newspapers and the public of New Orleans to exercise patience and fairness Surely an engineer of such high reputation as Mr. Harrod is entitled to the presumption which the law extends to the meanest criminal. He deserves to be considered innocent until he is proved guilty. We should assume that he has acted from honest and honorable motives until the contrary is proved.

There is no doubt, of course, that Mr. Harrod had the power to exact a price concession from the contractor for the privilege of using the cheaper cement, and very likely he erred in not doing this. On the other hand, it is quite within the possibilities that it was the part of wisdom and good engineering not to exact such a concession.

As a rule good and satisfactory work is only to be secured from a contractor who is making both ends meet, with a margin besides. Engineers very commonly keep a record of the material and time accounts of contractors, and thus have a close check on the profits they are making—or are not making. It is not uncommon for a contractor who is trying to do good and honest work, but who is losing money on his job, to be helped out by the engineer when some alteration of the letter of the specifications is possible without detriment to the work. Who shall say that such a course is not wise and sound business management? A contrary course would mean usually the failure of the contractor and consequent delay and extra expense, or else the scamping of the work by the contractor in ways which the best inspection cannot prevent.

We write entirely without knowledge of the actual conditions and merely to show the possibilities of the case, in order that it may not be prejudged. The point we would emphasize is that only after a judicial determination of the actual facts is anyone entitled to pass judgment upon Mr. Harrod's conduct. It appears likely that such a determination may be made, as the Drainage Commission has declared its intention of bringing suit against the contractor to recover the amount which he saved by the use of the cheaper cement.

It will be noted that in his statement printed above Mr. Harrod, with a frankness which certainly does him credit, estimates the saving to the contractor through the use of the cheaper cement at over $31,000. He bases this estimate, however, on the prices bid for the various cements in 1901. Now, if we mistake not, the demand for cement in this country during the years from 1896 to 1900 was so great that home manufacturers were very stiff in their prices. It is quite possible that the difference in price of foreign and American cement at New Orleans during that time was less than in 1901, when American production of Portland cement had so increased that manufacturers were hustling for orders instead of sitting in their offices waiting for buyers to come to them. If this view of the case is correct, the amount which the contractors actually saved by the use of the cheaper cement may be much less than the sum which Mr. Harrod estimated.

Finally, we hardly need point out that the New Orleans situation may well be a warning to every engineer that no matter how high his reputation or how honorable his motives, he cannot be too circumspect in the discharge of his trusts. Where the necessity arises for departing from specifications or granting privileges to contractors, the approval of the authority placed over the engineer will act as a bar to future charges of favoritism or corrupt collusion. Many an honorable and high-minded engineer has suffered grievous injury through such charges, which are too often originated in personal jealousy and spread broadcast to further the ends of political partisanship. That the engineering profession should support its members in such an emergency is self-evident, and is illustrated, we are pleased to add, by the action which many of the engineers of New Orleans have taken in defending Mr. Harrod from the attacks in the public press.

LETTERS TO THE EDITOR.

Concerning Formulas for Computing Probable Errors.

Sir: Prof. Fuertes has an interesting paper on "Short and Easy Methods for Computing Probable Errors" in Vol. 46 of the Transactions of the American Society of Civil Engineers. This paper did not reach the writer for discussion, and as the formulas therein given are in text-books of Least Squares, and do not claim originality, perhaps discussion was not necessary. It is not quite plain, however, whether Prof. Fuertes is claiming that Mr. F. D. Covarrubias is the originator of the method of abbreviation or whether he merely instances his use of it under the peculiar conditions cited.

Prof. Fuertes' formulas (1) and (2) are the same as those in Prof. Merriman's text-book, Nos. (35) and (36). Their originator, as mentioned by Prof. Merriman in another publication, is Dr. Charles A. F. Peters. Peters' formulas appeared in the "Astronomische Nachrichten," Vol. 44, and were first brought to attention in this country by Schott in the U. S. Coast Survey Report for 1856. This must have been some ten or twelve years previous to the use of the formulas by Covarrubias in Mexico.

The formula in its simplified condition, as given by Prof. Fuertes in a foot-note, was used by Bessel in 1816.

A word of caution may be needed in regard to the employment of the formulas given by Prof. Fuertes. When the residuals are few in number, or are not very accordant, it would be well for the computer to use the formulas generally employed, involving the small extra labor of squaring the residuals.

For example, if two series of observations give the residuals:

5 6 2 7 3 8 10 9 3 5,

and

14 3 0 6 20 2 0 2 1 10,

respectively; Peters' formulas will give $r = 5.17$, and $r_0 = 1.63$ in both cases, while the more rigorous method of squares will give:

$r = 4.51$, $r_0 = 1.43$ in the first series;

and

$r = 6.16$, $r_0 = 1.95$ in the second.

In the comparison here given it is evident from inspection merely that the probable errors of the individual observations and of their mean in the first series should be less than in the second, while the Peters (Covarrubias?) formulas would make them alike in both.

Very truly yours, Horace Andrews.

51 State St., Albany, N. Y., Feb. 11, 1902.

A Comparison of Two Recent Specifications for Proportioning Concrete.

Sir: Upon reading the article entitled "Concrete and Brick Reservoir at Canton, Ill.," which appeared on p. 129 of the Feb. 13, 1902, number of Engineering News, and comparing it with the concrete requirements of the New York Rapid Transit Railway on the opposite page, I came to the conclusion that there is a great diversity in the proportions of the ingredients of the concrete as made at the two points. At Canton, Ill., the theoretical proportions were fixed at 1 cu. ft. Portland cement, 3⅛ cu. ft. of sand and 8⅛ cu. ft. of crushed rock, predicated upon 40% of voids in the crushed rock and 30% in the sand. By adding 10%, this proportion is modified to 1, 3½ and 7½.

The requirements of the Rapid Transit Railway for the concrete in arches are 1 portion Portland cement, 2 portions sand and 4 portions stone. Making an allowance of 10%, these theoretical proportions would be 1, 2 and 4.4, corresponding to 45% voids in the stone and 50% in the sand. For the other parts of the work, the requirements are 1, 2½ and 5, or, with an allowance of 10%, 1, 2⅞ and 5½, corresponding to 45% voids in the stone and 40% in

the sand. Assuming that the crushed stone is limestone, which will probably weigh 166⅔ lbs. per cu. ft. This would make the Canton, Ill., crushed rock weigh 100 lbs.

The sand is stated to weigh 100 lbs. per bushel, equal to 80 lbs. per cu. ft., and is said to have contained only 30% of voids, which would make a solid cu. ft. weigh but 114 lbs., certainly a very light character of stone.

In looking over the cost per cu. yd., I note that the quantity of cement is given as 0.586 barrel, evidently a transposition of figures. I think it should be 0.856. From this same data I have constructed the following table:

			Proportion.
*Rock.....	23.14 cu. ft. at 100 lbs.	= 2,314 lbs.	7.50
Sand.....	12.52 " " " 80 "	= 325 "	4.06
†Cement.....	3.08 " " " 105.5 "	= 1,010 "	1.00
Totals =	38.74 " "	= 3,649 "	

*Medium crushed rock.
†American Portland cement.

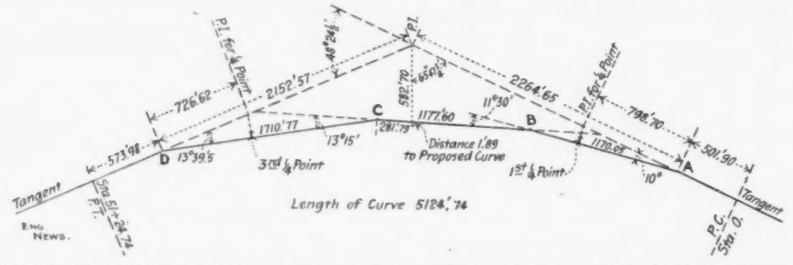
This would make the concrete weigh but 135 lbs. per cu. ft. and shows a reduction of the materials of 70%. The peculiarity of this table is that it shows an excess of 22% of sand over that specified to have been used, or in other words the mortar is made up in the proportion of 1 part cement to 4 parts of sand. The voids in the stone and in the sand are certainly extremely low and it would be interesting to know of the character of these materials and whether they were wet or dry when the voids were determined. Respectfully,

Emile Low, M. Am. Soc. C. E.

498 Ashland Ave., Buffalo, N. Y., Feb. 17, 1902.

Relining of Railroad Curves.

Sir: The article in the issue of Dec 26, 1901, on relining of railroad curves is of considerable interest to maintenance of way engineers. The cardinal principle in curve relining is to get a curve that will fit the track as near as possible. Unnecessary throwing of track means extra cost. Often a bridge or narrow shoulder determines what kind of a curve to use. The consideration of a fractional minute in the degree of curve should not be a governing



point at all. The sketch herewith is a case from actual practice. The curve lies on a double-track roadbed on a fill. Dense timber is found on both sides of the track. To have run out the P. I. would have required enormous labor. The two tangents were connected by lines running from A to D, as shown. These lines were so taken as to lie almost entirely on the roadbed. This made careful measurement of distances possible. The successive angles and distances were then measured. These measurements, together with the distance from line B C along external to center of track, were the only data taken for the curve.

The remaining figures (together with some more not shown) were computed.

The P. C., first ¼ point, middle point, third ¼ point and P. T. were all located from these lines before any attempt was made to run in the curve. These five points gave a good idea as to the fitness of the line. For their position could easily be changed by simply changing the middle point. Before running in the curve these points were tested for deflection by covering each successive point with the transit, starting at the P. C. The curve was then run in by starting at the P. C., using the successive points ahead as foresights. As to accuracy, the ¼ points and middle points checked with 0.03 ft. for distance, as measured along the curve. The P. T. fell short 0.24 ft. This is very close work for a curve of this length. The figuring may seem a great task, but it is better to figure than to cut brush. It took 3½ hours to do the figuring.

The degree of this particular curve was made 56.676′. To avoid the turning off of fractional minutes in deflection, the head chainman was instructed to set the full stations at 98.80 ft. instead of 100 ft. The curve was then run in as a 58′ curve.

In conclusion, the writer believes the following is good practice:

First. Keep in mind the cost of the proposed improvements. Extra gangs are expensive bodies.

Second. Make the field operations as few and simple as possible, no matter how complicated the figuring.

Third. Always establish a foresight. The old way of running in a curve by backsight is the cause of the so-called "fudging" at the P. T. end of the curve.

B. R. Leffler.

Engineer's Office, C., B. & Q. Ry., Burlington, Ia., Feb. 4, 1902.

Priming Centrifugal Mine Pumps Handling Muddy Water.

Sir: In your issue of Jan. 23, 1902, I note an article entitled "A Mine Pumping Plant With Electrically Driven Centrifugal Pumps."

We are phosphate miners, using centrifugal pumps to lift from the mines our output, a mixture of water, sand, gravel and mud. The proportions are immaterial, but to give you an idea we will say the mixture is: 90% water, 3% sand, 3% gravel, 4% clay, a total of 100%.

As the work progresses these centrifugals are moved ahead once or twice a month. They are driven by steam engines, direct or belt connected, steam being carried in pipes. We have considered electricity, but have not been able to look favorably upon it because it fails to furnish any method of priming the centrifugal.

The method now used is a steam ejector placed at the highest point in the pump. Steam is run through the ejector, and on stopping up the discharge end of the pump the air is exhausted from the suction pipe and water follows till it is discharged through ejector. Thus the pump is primed and ready for the engine to be started.

The work requires this priming to be done frequently during the day, and with long lengths of large pipe to be exhausted, the ejectors require a 2-in. steam pipe in order to quickly do the priming. No foot valve can be used, for in a shut-down the solid matter settles on the valve and it cannot be opened again. An electrically operated air pump to take the place of ejector will not answer because it will eventually fill with water full of grit, and the cylinders will soon cut, so as to ruin the efficiency of the pump. A water ejector in place of steam does not seem to work and we can learn of no other methods.

Can you tell us where to look for help in this matter? We judge the Sulzer pumps have foot valves, though the article does not intimate how they are primed.

Yours truly, B. H. Heyward.

Sand Pebble Phosphate Co., Pebble, Fla., Feb. 12, 1902.

(The installation of Sulzer centrifugal pumps at Horcajo, referred to in the above letter, comprises a check valve in each discharge pipe leading from the pump at one level to the one next above, and probably also a foot valve in the suction pipe. The lowest, or suction pump, and the suction pipe, are primed either by means of a hand-pump provided for that purpose, or by water drawn from the discharge pipe leading to the level next above. The water carries only an ordinary amount of silt. The conditions imposed by the nature of the material to be handled in the case presented by our correspondent, make the problem rather difficult. It is possible that a small tank or reservoir might be used to prime the pump, being itself charged by the pump while that is running. We should be pleased to hear from any of our readers who may have encountered and overcome difficulties like those described in the above letter.—Ed.)

A Proposed Four-Track Tunnel for Lower Broadway.

Sir: The apparent determination of the Rapid Transit Commission to connect the four-track tunnel of the Manhattan system with the proposed Brooklyn tunnel under the East River by only two tracks under lower Broadway is open to serious objection. It must be remembered that this connecting two-track tunnel will have to carry the traffic of the Manhattan system below City Hall, as well as the traffic of the Brooklyn tunnel, both of which the writer believes will be large enough to justify separate tracks in order to prevent congestion. Another objection to the commission's plan, and one which may cause further delay and possible danger, is the proposed switching system where the Brooklyn tracks connect with the Manhattan tracks in the neighborhood of Bowling Green. Any railway engineer will appreciate the difficulty of trying to pass a heavy train traffic through such a point in tunnels.

It is to be presumed that the commission, having the above objections in mind, had weighty reasons for proposing a two-track tunnel where a four-track tunnel seemed so obviously needed. Four tracks on a level in tunnel with "island" platforms at the proposed stations at Fulton, Wall and Bowling Green, are impracticable under a narrow street like Broadway, for the reason that such a tunnel would require a width of 80 ft., while the width from area line to area line is only 66 ft.

The writer's plan of a double-decked four-tracked tunnel is free from the objections of the wide tunnel and has many advantages over other forms of construction which have been proposed. Fig. 1 shows a cross section of the writer's plan, in which it will be noted that the base of the rail of the lower tunnel is only 30 ft. below the surface of the street. This economy in depth is obtained by setting the yokes of the electric conduit in a concrete filling between the channel beams which form the roof of the tunnel, as shown in Fig. 1. The drain channels of the conduit are formed of a flat tray which rests between the roof beams of the tunnel as shown. These features of the double-decked tunnel are suggested in order to save every inch in depth possible. By this construction the proposed tunnel floor will be safely above the water line until Bowling Green is reached. At this point the tunnel will be bottomed on bed rock and the width of the street here precludes the possibility of damage to building foundations. From Bowling Green to South Ferry there are no engineering obstacles to the construction of the writer's plan of tunnel. At South Ferry this plan lends itself admirably to the conditions. Here the tracks in the upper, or Manhattan tunnel, form a loop, while the lower tracks

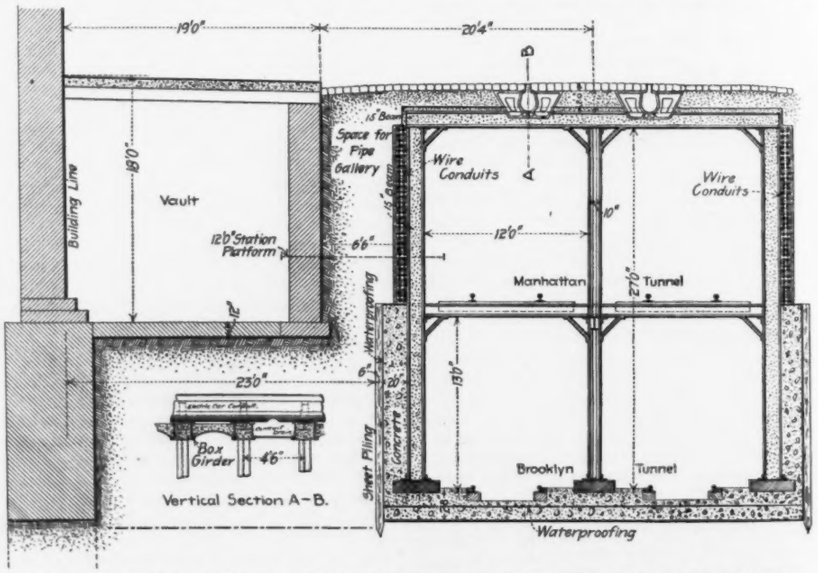


FIG. 1. SECTION OF DOUBLE-DECK FOUR-TRACK TUNNEL PROPOSED FOR LOWER BROADWAY.

J. W. Reno, Designer.

are at the proper depth to continue on a 2% grade under the East River to Brooklyn.

Heretofore the commission has on several occasions objected to the Reno plan on account of the possibility of noise from the upper tracks. The writer has given considerable thought to this objection and suggests a noiseless floor system shown in longitudinal section in Fig. 2. This floor consists of transverse steel trough girders, in the troughs of which are set sawed oak ties to which the rails are spiked. The ties are insulated from the steel girders by a filling of asphaltic composition which is also carried up so as to enclose the base of the rail. It is believed that this construction will cut off all vibration and make a practically noiseless track for the upper trains.

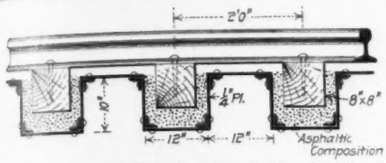


Fig. 2. Proposed Noiseless Track Construction for Upper Deck of Double-Deck Tunnel.

The commission has also objected to the Reno plan in the past on account of the depth at one time suggested below the surface and the consequent danger to building foundations. The great economy in depth attained by the present plan, and the fact that the heavier buildings on lower Broadway all go below the floor of the tunnel, will remove this objection. As now proposed, the depth of 30 ft. for the floor of the tunnel will enable the tunnel walls to be constructed in dry ground at a horizontal distance of from 23 ft. to 26 ft. from the building line, thus leaving adequate space for safe excavation.

The tunnel walls are composed of vertical steel I-beams set 4½ ft. apart with a concrete filling between them, as shown in Fig. 1. They will be built one at a time in a narrow trench 3½ ft. wide; sunk in short sections between a double row of sheet piling previously driven. The core of ground will not be removed until the tunnel walls and roof are finished, and then only as far down as the middle line of the tunnel where the transverse beams can be put in. When these beam braces are in position the tunnel core will be removed gradually to the point where the invert may be laid. Thus no excavation need be made until the space between the walls is permanently braced. The foundations for the central columns will be constructed by sloping the core down from each sidewall to the center of the floor of the tunnel and laying a strip of concrete along this central line. The final operation will consist in removing the remainder of the core, one side at a time, down to the floor line, and then laying the remainder of the concrete flooring. Yours truly,

Jesse W. Reno, E. M.

351 West 35th St., New York, Feb. 24, 1902.

By-product Charcoal Plant of the Lake Superior Power Co.

Sir: I have read with much interest the article on "The Engineering and Industrial Enterprises at Sault Ste. Marie," in your issue of Jan. 9. The said article has a great many true and well-presented facts, which no doubt will attract the notice of many of your readers, but I beg to draw your attention to one particular inaccuracy, namely, the description of our by-product charcoal plant. It is true that Mr. Daube directed the construction of the experimental plant described for the demonstration of certain results claimed for the so-called "Economic Process," but the result obtained proved the process a failure and consequently it was not adopted.

The charcoal plant decided on and now under construction, near the steel plant, will carbonize the wood by two different methods; namely, that in beehive kilns and that in steel retorts, both with the recovery of the by-products.

The retort plant will consist of 20 oven retorts, of 8 cords capacity, each 46 ft. long, 6 ft. 3 ins. wide and 8 ft. 4 ins. high. The retorts are set between brick walls in a way similar to the setting of a steam boiler. A track of standard gage runs through each retort. The wood, placed on iron cars holding 2 cords each, is run into the retorts, the doors at each end are closed, fires are started under the shell, and the wood thus carbonized. The gases from the carbonizing wood are drawn off as rapidly as formed, condensed and delivered to the by-product building for further treatment. In 18 to 24 hours the wood is carbonized and is then removed to an iron cooler of the same size and shape as the retort. In this cooler the wood remains 24 hours and is then drawn into a second cooler, where it remains 24 hours longer, being then thoroughly cooled and ready for the blast furnace. The charcoal, still on the same iron cars on which it entered the retort as wood, is then taken by a steam engine to the furnace, where it is used as required. The process is continuous, for as soon as a retort or cooler is emptied it is immediately refilled.

The advantage of this process is the small amount of handling given to the wood and charcoal, no manual labor being required after the cars are filled with wood until they are emptied of the charcoal. It also gives the highest yield of by-product. This plant has a capacity of 160 cords per day, yielding about 8,000 bushels of charcoal.

The kiln plant will consist of 56 kilns 30 ft. in diameter at the bottom, 28 ft. at spring of arch, 14 ft. high to spring of arch, and 5 ft. to top of arch, each kiln holding about 65 cords of wood. These kilns will be arranged in four batteries of 14 kilns each. The gases will be drawn from the kilns as formed and forced through a series of copper condensers from which the condensed product passes to the by-product plant and the combustible gases are led to the boilers. The charcoal will be forked into cars, operated somewhat upon the cable system used for street cars, by which it will be carried to the furnace at any speed desired. This plant will have a daily capacity of 150 cords of wood, yielding about 7,000 bushels of charcoal.

In the by-product building will be recovered the wood alcohol (about 100,000 gallons, 82% strength, per month), the acetate of lime (about 750 tons per month) and various distillates of wood tar, such as creosote, guaiacol, etc. For the handling of this liquor there will be a boiler plant consisting of 2,400 HP. (Stirling) boilers, to be increased to 5,000 HP. when complete plans are finished. There will also be a pumping station of 10,000,000 gallons daily capacity.

The place and method of carbonization to be adopted for the remaining 300 cords will be determined upon later on. The product from the same will all be shipped by rail to the works; the charcoal in specially designed cars to the blast furnaces, and the by-product liquors in tank cars to the by-product works described, which are capable of handling the product from 600 cords of wood per day. Yours truly,

Ernst A. Sjostedt,

Chief Metallurgist, Lake Superior Power Co.

Sault Ste. Marie, Ont., Canada, Jan. 25, 1902.

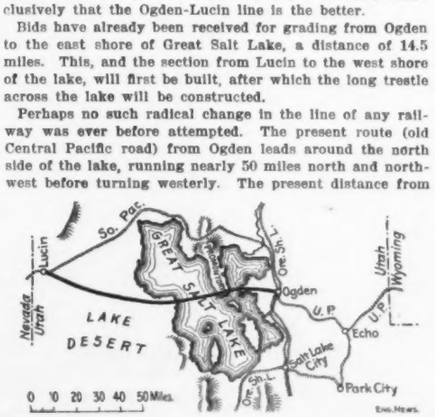
(We may note that the member of the editorial staff who visited the Sault Ste. Marie works was introduced to Mr. Daube in the office of the Lake Superior Power Co., examined the experimental plant with him, and understood that this system was to be adopted. The description of Mr. Daube's system is correct, but it seems that the system has not been adopted, for reasons given in the above letter.—Ed.)

The New Line of the Southern Pacific Ry. Across Great Salt Lake.

Sir: The Ogden-Lucin cut-off of the Southern Pacific Ry., which has been in contemplation for the past two or three years, is at last to be built. Officers of the Southern Pacific and Union Pacific companies were at Salt Lake City on Feb. 10, and met a committee of the Salt Lake Commercial Club, which presented arguments in favor of a cut-off from Lucin by way of the south shore of Great Salt Lake to Salt Lake City. The committee claimed such a line would be cheaper to build and operate than the Ogden-Lucin line; but the railway officials showed conclusively that the Ogden-Lucin line is the better.

Bids have already been received for grading from Ogden to the east shore of Great Salt Lake, a distance of 14.5 miles. This, and the section from Lucin to the west shore of the lake, will first be built, after which the long trestle across the lake will be constructed.

Perhaps no such radical change in the line of any railway was ever before attempted. The present route (old Central Pacific road) from Ogden leads around the north side of the lake, running nearly 50 miles north and northwest before turning westerly. The present distance from Ogden to Lucin is 145.5 miles. More objectionable even than the increase of distance due to this long detour or than the many curves are the heavy grades required in getting over the Promontory hill and the Kelton hill. The elevation of the former summit is 4,905 ft. and of the latter 4,700, though other parts of the line, but a few miles distant, are but little above the surface of the lake, which is 4,200 ft. A shore line along the north side of the lake is out of the question, on account of the extreme irregularity and length of the same, as a glance at the map will show. The projected cut-off will run a little north of west from Ogden to the extremity of a cape projecting from the east shore. From here a trestle seven miles long will lead to the south end of the peninsula, known as the Promontory. The line then leads across this peninsula for about five miles and then passes by a trestle 18 miles long across the main body of the lake to Strong's Knob, a projection from the west shore.



Map of Great Salt Lake, Showing Old and New Line of Southern Pacific Co., from Ogden to Lucin.

The deepest water (30 ft.) is encountered on this last section of the line. Both trestles will be on tangents. From Strong's Knob the road will run a little north of west to Lucin, 58 miles distant. The total length of the cut-off will be 102.5 miles, a saving of 43 miles over the present route. Curves will be few and light. The fall from Ogden to the east shore is 100 ft., and the rise from Strong's Knob to Lucin is 500 ft. in 58 miles. Both of these allow of very light grades, the country being quite uniform. A bridge will be required over the Weber River at Ogden. Just what type of construction will be adopted for the trestles has not yet been made public.

Chief Engineer Hood, in an interview, said that they would be made high enough to allow of a rise of 5 or 6 ft. in the present surface of the lake, and that they could then be raised 10 ft. higher if necessary. The extremely low stage of water in the lake now, which has continued for nearly two years, makes the present a peculiarly favorable time for the surveys and construction of the trestle. The experience at the bathing resorts on the lake has been that the sand tends to accumulate around piles driven in the bottom. If the same experience is had with the pile supports of the trestle, the result will be a rapid shallowing of water along the same, giving an increased security for the route as time progresses.

The Southern Pacific engineering department last year made very thorough surveys for a cut-off around the south shore of the lake, but found that a line by that route would be 67 miles longer from Ogden to Lucin than the adopted route across the lake, and would also have some heavy grades. It is officially stated that the cost of transporting the 600,000 tons of through freight that yearly pass over the Union Pacific and Southern Pacific would be $1.50 per ton more by the south shore cut-off than by the lake cut-off.

The great reduction in grades, curvature and distance that has been made on the Union Pacific's Wyoming lines during the past two or three years is to be followed during this year and next by radical changes on the Southern Pacific. The cut-off across Great Salt Lake, and the reconstruction of much of the line in Nevada are to be carried out to enable through traffic to be economically handled and heavy and fast trains to be moved.

Yours truly, W. P. H.

Salt Lake City, Utah, Feb. 13, 1902.

The Brooklyn Bridge Terminal.

Sir: So much is being said nowadays (principally in the daily papers) about the congestion of the trolley traffic in the evening at the New York terminal of the Brooklyn Bridge, that I hope to be pardoned if I add a little to the discussion. My opportunities for observing the conditions were very favorable during the examination of the bridge last summer. I doubt if anything I have to say on the subject is original, but I have never seen the various parts collected into a whole.

It is evident that the way to carry the greatest number of passengers over the bridge is to run as many cars as possible past any point in a given time, and that any feature which causes more than the average delay tends to limit the capacity of the whole system. Also, that at any period no increase of capacity can be gained by improving any feature beyond the carrying limit of that causing the greatest average delay during the round trip.

As the cars are now run, without transfer of passengers on the Brooklyn end of the bridge, the round trip is divided into five parts: the approach of the nearly empty car to the bridge, its journey across the north roadway, its passage around the loop where its passengers are received, its run to Brooklyn on the south roadway, and finally its journey through the streets of Brooklyn to the homes of the passengers.

No serious delays occur because of the trip over the north roadway, as this roadway is nearly free from teams in the evening. The trip over the south roadway, however, is attended with numerous delays, the congestion of teams here being nearly coincident with that of the trolley cars. Most of these teams are without loads and are returning to Brooklyn at the end of the day's work. Many of them wish to drive faster than others, the result being that the faster teams are continually turning into the trolley track in order to be able to pass the slower ones. When once on the track, the team can not leave it until a break occurs in the line of teams to the south. Delays on the south roadway from teams could be prevented by placing a high and substantial guard rail just south of the track, between the cars and the teams. Sections of the guard rail could be made removable and several openings could, if advisable, be left during the middle of the day. This guard rail would be a source of annoyance to the faster teams, but a much less serious one than ruling all teams off the bridge during the evening rush hours. Doing the latter is practically equivalent to ruling them off the bridge entirely, as teams, like passengers,

can in very few instances choose the time at which they return to Brooklyn.

The surface loops at the New York terminal are sources of both delay and danger, the latter very largely because several lines of cars use each loop and passengers wishing to reach any particular car have to push their way through a crowd of other passengers standing next the car who do not wish to take it. Both the delay and danger can be obviated by elevating the loops and giving separate access to each side of each track from below. A separate loop, or a separate part of a loop, should also be provided for each line of cars.

The most important source of delay, however, seems to be the congestion of traffic in the Brooklyn streets during the evening rush hours. While some other streets are used, most of the trolley cars running on the bridge seem to use Fulton St., as do most of the cars connecting with Fulton Ferry. The congestion on Fulton St., between the City Hall and Flatbush Ave., opposite the large department stories, is very great during the rush hours of the evening, and it is probably that feature which at present limits the capacity of the bridge. There is also much congestion on lower Fulton St., between the City Hall and the bridge. The cars run close together, by jerks, often stopping every few feet, and when a delay occurs to any car, most of the following cars have to stop also.

It is extremely doubtful if the trolley car capacity of the Brooklyn Bridge can be materially increased in any way except by changing many of the cars now using Fulton St. to some less crowded thoroughfare, or by transferring passengers to other cars at the Brooklyn end of the bridge. The former method will get the passengers to their homes in a shorter time, while the latter would merely relieve the present congested conditions at the New York terminal. Much of the crowding and danger at the New York terminal could be removed by elevating the loops and separating the different car lines, though with other conditions, as at present, this would probably add very little to the capacity of the system as a whole.

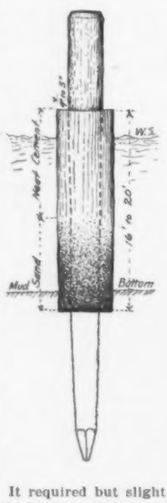
Edwin Duryea, Jr., C. E.

No. 1 Broadway, New York city, Feb. 19, 1902.

Protecting Piles by Cement Grouting; Flushed and Grouted Masonry Joints.

Sir: Under the caption of "The Comparative Value of Flushed and Grouted Masonry Joints for Dams and Other Structures," which appeared in your issue of Feb. 6, an invitation is extended in an appended editorial note for an expression of the opinions of others on this subject. I contribute the following account of some grouting that came under my observation while engaged on a survey of a part of the harbor of Pensacola, Fla.

A large dock about one-half mile in length extended into the harbor and was supported by cypress piling. The piles (see sketch) were covered by a coating from 4 to 5 ins. in thickness, consisting of hydraulic cement and sand (brand and proportion not known). Several of these piles had been removed, and, upon examination, the sand appeared to have settled towards the bottom of the piles, leaving the neat cement at the top. These cement coverings were intended to protect the piles from the ravages of the "teredo navalis," and they seemed to have successfully accomplished the purpose for which they were intended.



The cement, according to reports, was deposited as follows: Wooden water-tight flasks were secured around the piles and were forced for some distance into the mud bottom. They were then pumped out and filled with cement grout, which was allowed to set before removing the flasks. These cement coverings were from 16 to 20 ft. long, through which length (or rather height), the sand would have time to settle to the bottom. It required but slight effort to break off large pieces from the bottoms of the coverings, which, upon examination, appeared to be nearly all sand, there not being sufficient cement to bond the sand thoroughly. The upper parts of the coverings were composed of neat cement, which required considerable force to fracture. Perhaps the flasks leaked, or else the grout was made too thin, allowing the sand, in either case, to settle to the bottom before the cement had time to harden.

Upon this subject of grouting, as upon many other important technical points, engineers seem to differ, and their opinions seem to be about equally divided between the two methods of laying large masses of masonry. The example just mentioned, however, seems to be an extreme case that would probably not occur where grout had to work its way through the voids of a pile of stone; the eddies formed by the flowing grout, in turning angles and filling voids, would assist in keeping it thoroughly mixed until the pouring had ceased.

Personally, I favor the use of flush mortar joints, and in "properly testing and instructing" masons, as Mr. W. D. Taylor describes in his communication in your issue of Feb. 6. I have found some of them, however, so thoroughly "set in their ways" that the threat of dismissal had no effect while the very large majority cheerfully obeyed instructions.

Yours truly, H. F. Wilson, Jr.

315 Chalifoux Building, Birmingham, Ala.,
Feb. 10, 1902.

Methods Used in Grouting the Upper Tallassee Dam on the Tallapoosa River.

Sir: I submit for the information of your readers a brief account of the methods used in grouting the Upper Tallassee Dam, at or near Tallassee, Ala.

The combined length of the dam and power house was about 900 ft. The cableway, stretched between hills at either end, was about 1,100 ft. long between towers. It was employed to convey material along the line of dam. At one end of the dam a platform, sand bins and cement house was erected. A gravity concrete mixer was located at the end of the platform under the line of cable. Here the cement and sand (1:2) were measured, run through the mixer dry, dropped into the grout boxes and finally conveyed by the cable to the station to be grouted. The dimensions of the boxes were 6 ft. 6 ins. long, 4 ft. wide on the bottom, 4 ft. 8 ins. on top, and 1 ft. 4 ins. high. The boxes had a slide door at one end.

The sizes of the sections to be grouted were determined by the cut-offs. They ranged from 8 × 10 ft. to 15 × 15 ft. in plan and were about 4 ft. high. The large stone were left extending above the grouted area, making a bond for the next layer. After the grout box was placed in position four men, using large hoes made for the purpose with holes cut in the blade, brought the mixture to a liquid grout, while another man gaged the water and tended the slide door, thus keeping the liquid in motion until it disappeared amongst the stone. The flow had such a large area and head that there was no chance for a confinement of air. Two grout boxes were used on a large cut-off. The cut-offs were made by selecting the best stone of a medium size, weighing from 3 to 4 tons, and using mortar enough to hold the grout and give it a chance to get into all sides. The up-stream side of the dam was laid in Atlas Portland cement. The down-stream side and top were made of Portland cement concrete. The brand of cement used on the grouted portions was Louisville. The stone were large and but a few pinners were used, all carefully washed. The masons employed in placing the stone were men of experience on this class of work, having been employed on other dams and power houses.

I have grouted the forebay work for five different power houses and not one of them has leaked. The only safe way of obtaining tight work in heavy rubble masonry is by grouting.

The section of roilway that went out was the exception and was due to the cement and concrete not having time to set before the big freshet.

Oscar H. Briggs,

50 Moore St., Providence, R. I., Feb. 22, 1902.

(We infer from the foregoing that Mr. Briggs was foreman in charge of the work described. —Ed.)

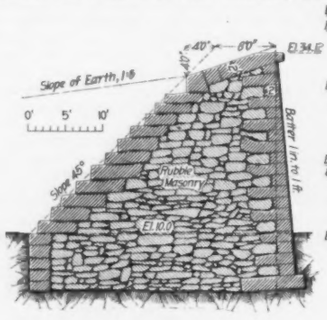


Fig. 1. Section of Dam of Essex Co., Lawrence, Mass.

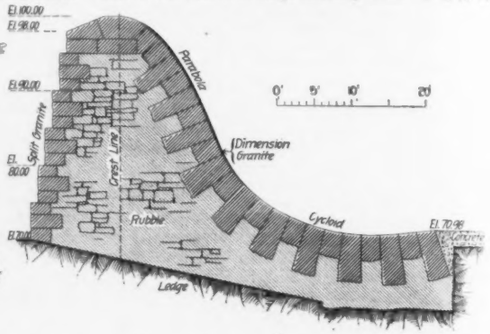


Fig. 2. Section of New Dam at Holyoke, Mass.
(Reproduced from Engineering News, Sept. 27, 1900.)

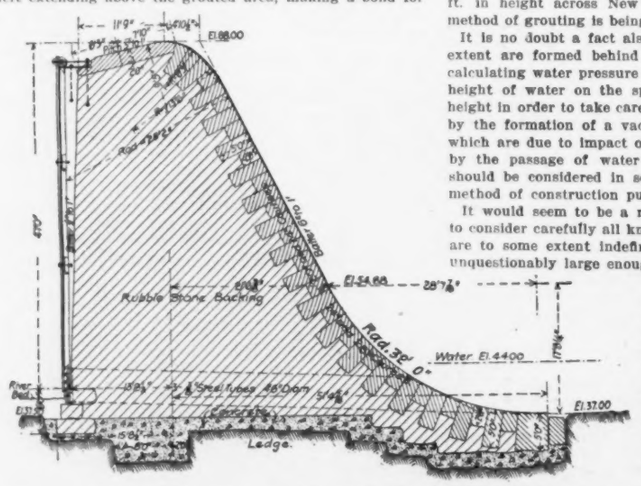


FIG. 3. RED BRIDGE DAM OF THE LUDLOW MFG. CO., ACROSS THE CHICOPEE RIVER.
(Reproduced from the "Engineering Record" for Dec. 28, 1901.)

Further Discussion on the Failure of Masonry Dams.

Sir: It is with considerable regret that the writer notices what seems to be a diversion of discussion from the points of main interest to the engineering profession with reference to the failure of large masonry dams. It is of course true that whether work should be grouted or whether it should be laid in mortar put in by hand, are important questions and deserve considerate and careful attention by the profession. They are not by any means new questions. Dams have been built in all these ways and they are still standing, and it is interesting to note that examples of both types suffered about equally at Tallassee. The writer is now engaged in the construction of one 40 ft. in height across New River, in Virginia, where the method of grouting is being pursued.

It is no doubt a fact also that vacuums to considerable extent are formed behind dams. Frizell recommends in calculating water pressure that one-third of the maximum height of water on the spillway be added to the actual height in order to take care of the stress in the dam caused by the formation of a vacuum behind it. Stresses occur which are due to impact of current and vibrations caused by the passage of water over the spillway. All these should be considered in selecting the designs and a safe method of construction pursued.

It would seem to be a requisite in engineering practice to consider carefully all known stresses and where stresses are to some extent indefinite to allow a factor of safety unquestionably large enough to cover them.

The writer has devoted some 12 years to bridge engineering and some six or eight years to the building of dams, and it seems to him that as large a factor of safety is necessary in one case as in the other. It is perfectly apparent that the shock to a structure from the passing of a railway train over a bridge is no greater, at any rate, than the passing of a heavy volume of water over the spillway of a dam.

It is unfortunate for any discussion of the integrity and character of classes of people to be indulged in. Human nature is practically the same in all classes and it is the experience and belief of the writer that it is good rather than bad. He knows that stone masons do, as a rule, a class of work their employers wish. Unless they are irritated by unreasonable and foolish exactions on the part of engineers they wish to carry out any intelligent instructions from the inspector.

A report of the situation at Portman Shoals, near Anderson, S. C., states that the work, which had been completed a few months ago broke along the lines of mortar

joints, but that the part of the old dam which was broken by being anchored to the new dam and by the passage of some 20 or 30 ft. of water over its crest, broke clean and smooth through the stone as well as cement. Blocks of stone containing more than a yard showed a break right through their centers. Age, of course, greatly strengthens masonry, but owners, engineers and contractors cannot risk failure during the first few years after construction.

It is not difficult to find good masonry or honest masons. It is the duty of every contractor to be personally and

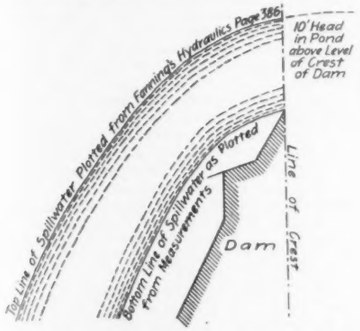

10' Head in Pond above Level of Crest of Dam
Top Line of Spillwater Plotted from Fanning's Hydraulics Page 386
Bottom Line of Spillwater as Plotted From Measurements
Line of Crest
Dam

Fig. 4. Flow of Water Over Dam of Columbus Power Co., Columbus, Ga.

financially responsible for the work of his men and it is the duty of the engineer to be professionally responsible for the work of the contractor. No good thing comes without care and labor, but with intelligent and reasonable attention good results are assured and it would be puerile for the engineers or the contractors to hide behind the masons.

There are just two types of dams which the engineering profession has agreed to be advisable. One is adapted to heads of less than 20 ft., or to conditions where the danger to the foundation from wear by the spill water can be neglected. This design would have its batter on the upstream side, sufficient width on top to withstand the stresses caused by the spillway and a batter of probably 2 ins. on the down-stream side in order to protect the foundation to some extent and to prevent undue crushing strain at the toe.

There appears on page 208, volume 44 of the Engineering News, a cut of the design of the dam of the Essex Co., at Lawrence, Mass., by Mr. Chas. S. Storrow, which is an excellent and safe illustration of the first type. (See Fig. 1.)

It is the recollection of the writer that practically the same design was presented by the late lamented Mr. John Hill, an engineer of note in Columbus, who was engineer for the Columbus Power Co. and part owner in that company. But for his death this type would probably have been used.

The second type, although probably preferable in all cases, is less economic in material for small heads, though necessary for large ones or where there is danger to the foundation from the effect of the spill water. In this case a rollway must be used of long enough slope to insure contact of the spill water on the stone work under all conditions.

Designs of two dams have appeared in the professional journals recently: One on page 208, volume 44, Engineer-

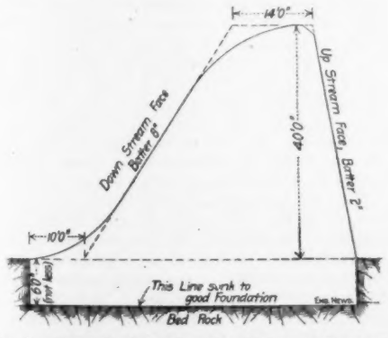


Fig. 5. Suggested Design for Masonry Dam with Extreme Spillway Heads of 15 to 18 ft.

ing News, of the New Holyoke Dam, designed to carry 4 ft. of water on the spillway, Fig. 2, and one (Fig. 3) in the "Engineering Record" of Dec. 28, 1901, showing the dam of the Ludlow Mfg. Co., designed to carry 6 ft. of water on its spillway. These two dams are excellent examples of intelligent and safe construction of the second type.

There is a third type, examples of which have recently failed, which is intermediate between these two. While it may be adapted to impounding water in reservoirs, it is not adapted to the foundation of a spillway in rivers of large drainage area and the only temptation to use it probably would come to an engineer from an unwritten demand by the company for which he is planning to hold expenditure within fixed points.

The dam referred to (in your issue of Feb. 6) near Athens, Ga., was no doubt filled in with mud and stands on the principle that the ordinary type of wooden or "gravity" dams stand. The excellence of the type carries in this case the burden of faulty detail of construction.

The other dam noted in your issue of Feb. 6, as being lawyer built, is interesting, in that it is referred to by the gentleman describing it as standing out of "the cussedness of inanimate things." An examination of this section shows it to be stable against overturning and the description of the method of construction by the use of "Beton Aglomere" seems to have been as intelligent and economic as the use of crude labor would permit. It is the same type as Mr. Storrow's dam, Fig. 1, and seems not to be exposed to overflow of any consequence.

Fig. 4 shows the unimpeded path of the water over the spillway of the dam that recently failed at Columbus, Ga., as plotted from observation made by Mr. W. S. Lee, Resident Engineer for the Columbus Power Co., with the water 10 ft. deep on the crest; also the theoretic path as worked out by Mr. Fanning. It will be noted that there is great coincidence between the two.

Fig. 5 shows a cut which the writer suggests being safe, if the depths on the spillway under extreme conditions are from 15 to 18 ft. He presents it not as anything original, but as a section which could not markedly be reduced without bringing the factor of safety too low to insure permanency.

B. H. Hardaway.

Columbus, Ga., Feb. 18, 1902.

Methods of Testing Fireproof Wood.

Sir: The article which appeared in your issue of Feb. 20, entitled, "A New Method of Testing Fire-Resisting Qualities of Fireproofed Wood," by Prof. Ira H. Woolson, is of very great interest to those of the engineering profession who have considered this problem.

During the past year, the writer has conducted a series of tests to determine the effect "fireproofing" has upon the strength of woods, and, in connection with tsese tests, has also made a number to ascertain their "fireproof" qualities. There can be no doubt on the question that the present methods of testing wood to determine these fireproof qualities are unsatisfactory, and more or less uncertain.

Professor Woolson points out very clearly some of the faults of the "shaving" test required by the Navy Department, and used also by the New York Building Department. Wood is not used in the shaving form, and this fact alone is very strongly against the usefulness of this test. The new method suggested by Professor Woolson is a decided improvement over the shaving test, but it seems to the writer that he has not carried his own arguments to their legitimate end. He rightly says that shavings are not used in building construction, and then he tests the fireproof qualities of wood on a sample 1 sq. in. in cross section.

The writer would go still farther, and endeavor more closely to reproduce actual conditions. Because of the peculiar characteristics of wood, as compared with iron, it is difficult to draw fair conclusions from so small a sample. Compared with wood, metal is very uniform, and yet we all know how advisable it is to reproduce, as far as possible, in our tests the conditions under which metal is to be used.

If we consider for a moment the way in which fire is likely to come in contact with wood in a structure, we find that it comes on one exposed surface only; the wood is backed up by some other material, or, at least, it is so placed that the flame generally gets access to one surface at a time. Remembering this, the writer took a piece of wood 4 ins. wide by 1 in. thick, stood it against a brick, and applied on one surface a Bunsen flame, using a known and definite air blast (20 lbs.). Then, after a definite exposure, sawing the piece in two, a comparison was made such as Professor Woolson makes with his small sticks, and the results differed quite materially from those obtained on sticks of 1-in. cross section. If preferred, a pyrometer could be used in this test.

If the solution injected into the wood to render it fireproof is of such a nature that, upon the application of heat, it seeks the air and generates a gas which repels fire, it is easy to understand how this will act more favorably in the shaving, or in a small stick exposed on all four sides to the flame, than it would if only one side were exposed. But is such a treatment the best for a fireproof wood? The more heat to which it is exposed, the more gas generated, and the less of the solution left in the stick. On such a specimen, a short-time test with much exposed surface, is eminently desirable. This brings us directly to the point which Professor Woolson makes as to a practical definition of "fireproof" wood, and the writer agrees with him that uniformity in this, as in the tests to be made, is extremely desirable. The tests finally adopted by the Building Department should be of such a nature that they can be carried on at the shops of the different companies fireproofing wood, rather than at a special laboratory; and from the writer's experience, it seems very desirable that strength tests should be conducted in connection with those to determine the fireproof qualities. There is certainly room for improvement when the navy specifications stipulate that fireproof wood must not deteriorate in strength more than 30%. If wood is properly fireproofed, deterioration to anything like 30% is unnecessary, and it would seem unwarranted to permit it.

It is to be hoped that at no distant date the suggestion of Professor Woolson will be carried out, regarding a commission to define fireproof wood, and to formulate an acceptable method of testing it. Not only do the recent fire calamities call for immediate action in this matter, but the effort now being made by the Forestry Board to stop the destruction of the forests impresses this necessity upon us. Very truly yours,

Geo. E. Mason.

University Heights, New York, Feb. 21, 1902.

Notes and Queries.

H. J. M., Brooklyn, N. Y., wishes the address of some engineer who can furnish plans for a siphon to discharge molten lead from a melting pot.

C. N., Jersey City, N. J., asks if the cause of the fall of an unfinished bridge span at Louisville, Ky., some years ago, was ever ascertained. He will find a full account of the accident, with an expert report on its cause, in our issue of Dec. 28, 1893.

THE PAN-FORMATION OF SOUTH AFRICA IN RELATION TO IRRIGATION PROBLEMS.

In a report by Mr. W. Willcocks, M. Inst. C. E., printed in the "Diamond Fields Advertiser," of Kimberly, S. A., the peculiar pan-formation of the northern portion of South Africa is discussed. The lower reaches of the Zand and Vet Rivers, and the Modder and Riet Rivers, traverse the undulating plains in valleys lying from 50 to 100 ft. below the general country level. Lying between these valleys and generally unconnected with them, are so-called "pans" or depressions. They are sometimes in connected groups, but as often quite independent of each other; and they vary from a few hundred square yards to 10 or 20 square miles in area. Some are 5 to 10 ft., and others are 100 ft. below the general surface. The soil about them is generally a red sandy loam, of light depth, with outcrops of sandstone or dolarite rock. Most of them are dry and seem impervious to water.

In this country, says Mr. Willcocks, 200,000 cu. ft. of stored water would irrigate one acre for a year. And each square mile of country could be depended upon for a runoff of 2,000,000 cu. ft. of water, even in a year of drought; or enough to irrigate ten acres. In ordinary years twice this amount can be safely secured, and canals could be constructed accordingly.

Mr. Willcocks suggests the use of these "pans" as reservoirs; to be filled from the river when in flood in January—April. A pan of 2 sq. miles in area and 25 ft. deep, would contain 700,000,000 cu. ft. of water; and allowing liberally for evaporation, the 500,000,000 cu. ft. left would irrigate 1,250 acres for a year. With the aid of pan-irrigation, 100,000 acres near Kimberly could be converted into first-class agricultural land.

SPECIFICATIONS FOR PAVING BRICK FOR USE AT WHEELING, W. VA.

We reprint below the specifications for paving brick on which bids for some 2,000,000 brick for use at Wheeling, W. Va., were recently invited. We are indebted to Mr. Chas. W. Lohse, City Engineer, for the specifications, regarding which he has written us as follows:

This city was among the first to use brick as paving material, and the specifications are the result of experience. You will note that only a single test is required, and the standard adopted is not extreme, as it was obtained from a series of tests made from brick delivered under last year's contract, and not from specially selected samples. Have omitted all tests which do not furnish a true indication of the value of the brick as regards its actual wear in the street, and which might increase the cost of manufacture without improving its worth as a paver.

With the exception of two short paragraphs of local interest, relating to delivery and form of proposal, the full specifications are as follows:

QUALITY.—The brick must be thoroughly hard, well and uniformly burned, and free from warps and fire cracks. They must be free from all loose lumps of uncrushed clay or laminations, and all particles completely fused and coalesced or vitrified. By "vitrified" is here meant a perfect blending of the constituents in burning. The quality requirements, compactness, uniformity of tex-

ture, hardness, toughness, vitrification must extend uniformly through the whole interior, and when broken, bricks shall show no variation in structure. Slight kiln marks or indentations will be no bar, but overburned or checked bricks will not be accepted.

DIMENSIONS.—The dimensions must be the same throughout the entire contract, and the size shall be approximately 8¼ × 4 × 2½ ins. This shall not be construed to shut out any brick of ordinary size. They shall be true in form and rectangular in shape.

TEST.—The bricks shall be subjected to such tests and inspection as the city engineer shall direct, but the governing test shall be the rattler test. For this test five entire bricks of each sample shall be placed in a tumbler or rattler of cylindrical form, 30 ins. in diameter, 4½ ft. in length, along with ten cast-iron weights of cylindrical form, weighing 6 lbs. each. These materials shall not be accompanied by any kind of cushioning material or any other material whatever. The rattler shall then be revolved for 30 minutes at a speed of 24 revs. per min. The bricks are weighed before they are placed in rattler and after they have been tested, and the loss in weight found. This last divided by the original weight gives the percentage of loss. The average percentage of loss of the five bricks shall not exceed 10%. In weighing the result of the rattler test, pieces of brick less than ½-lb. in weight shall be considered a normal loss, but in case of a single piece of brick, weighing more than ½-lb., the brick sustaining such loss may be discarded, and the percentage determined by using the remaining four bricks. This shall not be construed as permitting the rejection of an inferior brick, but as an assumption that brick had large fire crack, overlooked at the start. A second test of five brick may be required at the discretion of the city engineer.

INSPECTION.—The brick will be inspected at the kiln by a representative of the board of public works, if the board so elects. Contractor must give two days' notice to the clerk of the board of public works when bricks are to be loaded or shipped. The brick shall again be subject to inspection and rejection during the laying of the pavement or any handling of the brick and all brick rejected will not be paid for.

SAMPLE BRICK.—Bidder must submit not less than ten samples of brick proposed to be furnished; five of these brick to be subject to the rattler test, and the remaining five, supplied by the successful bidder, being retained as samples for comparison until the expiration of the contract, and all shipments shall be equal to samples.

Those interested in the difference between the rattler test as here described, and as recommended by a committee of the American Society of Municipal Improvements, last October, will find the latter in our issue of Oct. 16, 1901. The committee's rattler test, it should be added, is substantially the same as the one finally adopted, in the latter part of 1901, by the Committee on Technical Investigation of the National Brick Manufacturers' Association, copies of the report on which, we understand, may be secured from Mr. T. A. Randall, Secretary of the Association, Indianapolis, Ind.

MUNICIPAL OWNERSHIP OF THE LONDON WATER-works, now in the hands of seven or eight private companies, is proposed by a bill introduced in Parliament by the general government. The proposed plan provides for the purchase of the present works by agreement, if possible, and where not possible by arbitration. The arbitrators, who are named in the bill, are given absolute authority to determine the purchase price of any or all the works, with no allowance for compulsory sale. The works are to pass into the hands of a water board on Jan. 1, 1903. The board is to be a composite body of 67 members and a chairman and vice-chairman in addition, elected by the foregoing. The members are to be chosen from the governing bodies of the numerous local authorities having jurisdiction in the area now served by the metropolitan water companies. The London County Council is to have ten representatives on the water board and the various other authorities will have either two members or one member, according to the stipulations of the bill. The total area covered by the proposed water district, according to the London "Contract Journal," is 566 sq. miles, on which there is a population of 5,895,000. The bill is being opposed by the London County Council, which has for a number of years past been working for control of the water-works in question.

OLD AND NEW METHODS OF SEWAGE PURIFICATION AT PAWTUCKET, R. I.

Since Dec. 1, 1904, the city of Pawtucket, R. I., has been purifying by intermittent filtration the sewage of that part of the city which drains into the Moshassuck River. Between Dec. 1, 1894, and Nov. 1, 1895, a total of 13 beds were put in operation, with a combined available area of 2.35 acres. Before the sewage goes to the beds it passes through two settling tanks, each 30 × 100 ft. in plan. The available depth of the tanks is 3 ft., thus giving a capacity of about 67,000 gallons each, or 134,000 gallons in all. The sewers are combined (carrying both house drainage and storm water), and a small corner of one of the tanks is partitioned off and serves as a chamber for diverting the storm flow from the beds when it exceeds the average daily flow, or any amount for which the automatic regulating gates are shut. The purification plant was described in detail by Mr. Geo. A. Carpenter, City Engineer of Pawtucket, in our issue of Jan. 2, 1896.

Since January, 1900, one of the settling tanks has been used for two extended periods as a septic tank, and since March of the same year experiments with bacterial filter beds have been made. In our issue of Dec. 20, 1900, we published an abstract of a paper read by Mr. Carpenter before the League of American Municipalities, in which he reviewed the first period of experimentation with the septic tank. In Mr. Carpenter's annual report for the year ending Sept. 30, 1901, he summarizes the work of both the septic tank and the bacteria beds. This summary we reprint herewith, omitting the table of detailed analyses, but including the table (I.) of percentages of purification effected by the septic tank, the bacteria beds, and the intermittent sand filters. As Mr. Carpenter states, the tank used "was shallower than a good septic tank should be." The extract from the report is as follows:

For a period of ten months, from Jan. 13, 1900, to Nov. 13, 1900, all the sewage of the Moshassuck River district was passed through this tank. The tank was then drawn down, the accumulated sludge was measured, loaded into barrows and wheeled to an adjacent field, where it was composted and used the following spring by a farmer for the fertilization of crops.

On Nov. 20, 1900, sewage was again turned through this tank, and it was again continuously used as a septic tank until Aug. 7, 1901, when the sewage was again drawn off, the accumulated sludge measured and taken from the tank and composted.

TABLE I.—Purification Effected by Septic Tank and Several Filters. (Parts per 100,000.)

	Ammonia—Free. Sewage.	Ammonia—Free. Effluent.	Ammonia—Free. Removed, %.	Ammonia—Albuminoid. Sewage.	Ammonia—Albuminoid. Effluent.	Ammonia—Albuminoid. Removed, %.	Oxygen consumed. Sewage.	Oxygen consumed. Effluent.	Oxygen consumed. Removed, %
Purification effected by septic tank	7.38	6.75	8.5	1.29	0.739	42.7	12.29	8.28	37.7
" " " " " and sand filters	7.38	3.30	55.3	1.29	0.1383	89.3	13.29	1.96	85.2
" " " " " filter No. 14	7.38	3.39	54.1	1.29	0.3622	71.9	13.29	3.76	71.7
" " " " " " No. 15	7.38	2.80	62.1	1.29	0.2941	77.2	13.29	2.99	77.5
" " " " " " No. 15-16	7.38	1.55	79.0	1.29	0.2236	82.7	13.29	2.29	82.8
" " " " " " No. 15-17	7.38	1.20	83.7	1.29	0.1798	86.1	13.29	1.97	85.2

The following comparison of results obtained from the two experiments are of interest:

Period of service as septic tank	Jan. 13 to Nov. 13, 1900.	Nov. 20, '00-Aug. 7, 1901.
Experiment lasted	10 mos.	8 m. 18 d.
No. gallons sewage through tank	27,166,000	29,420,303
Sludge found in the tank at the end of the experiment, cu. yds.	147.48	142.58
Sludge pr million galls. sewage, yds.	5.428	4.846
Moisture of this sludge, %	81.75	79.11
(b) Equivalent number of cu.yds. of dry matter found in tank per million gallons of sewage	0.99	1.012
Solid dry matter, per million galls. contained in the sewage which passed into tank,* cu. yds.	5.023	4.577
Solid dry matter per million galls. which passed out of septic tank in the effluent,* cu. yds.	3.155	2.936
(a) Dry solid matter per million gallons left in tank, as shown by difference in last two sets of figures, cu. yds.	1.868	1.641
Solid matter transformed or septicised per million gallons, during passage of sewage through septic tank, as shown by difference between (a) and (b), cu. yds.	0.878	0.629
Solid matter septicised by the septic tank, %	47.0	38.3
Sludge per million gallons taken from sludge beds in the 6 years plant has been an intermittent filtration plant, cu. yds.	3.14	.....
Sludge per million gallons sewage now being taken from sewage on sludge beds, cu. yds.	4.87	.....
Albuminoid ammonia removed by septic tank, %	41.3	42.7
Albuminoid ammonia removed by sedimentation, %	.....	37.7

*As computed from the average results of analyses for "residue on evaporation."

CONTACT BEDS NOS. 14 AND 15.

As stated in the report of last year, two contact beds, each 15 × 15.5 × 4 ft. deep, were constructed to be operated in connection with the septic tank. Two rows of 2-in. tile underdrains placed 5 ft. apart were laid on the bottom of each bed and were covered with clean gravel of assorted sizes to a depth of 8 ins. On top of this gravel in bed No. 14, 3 ft. 4 ins. of crushed stone of the size known as chestnut were placed. In bed No. 15, 3 ft. 4 ins. of soft coal cinders which would pass through a screen of ¾-in mesh were deposited.

The effluent from the septic tank was turned on bed No. 14 at the rate of 509,000 gallons per acre from March 12, 1900, to Oct. 1, 1900. From Oct. 1, 1900, to Oct. 1, 1901, the rate has been 400,000 gallons per day. On the basis of 365 days in the year these rates become respectively 433,333 and 394,416 gallons per acre per day.

This septic effluent was also run on bed No. 15 at a rate of 526,000 gallons per acre until Oct. 1, 1901. From March 12, 1900, to Oct. 1, 1900, this was equivalent to 445,185 gallons per acre per day for 365 days in the year, and since that time the equivalent has been 414,814 gallons per acre per day for the full year. This reduction in rate per day, on the basis of 7 days in the week, is accounted for by the periods of rest which have been given these beds. Neither bed is dosed on Sunday and they were both rested for a period of eleven days in August, 1901. Both beds have also had several short rest periods, of one or two days, on several occasions during the year.

Careful measurements have shown that both of these beds have lost considerable of their original capacity. This has been due in part to a breaking down of the materials of which they are composed, to a settling together and to a gradual silting up.

In bed No. 14 the reduction in capacity due to settling together has been 11.1% and in bed No. 15, 21.7%. The loss of capacity due to silting up from the retention of mineral and organic matter contained in the septic effluent has been 18.83% in the case of bed No. 14, and 22.48% for bed No. 15. It is thus seen that the total loss of capacity in bed No. 14 has been about 30% and in bed No. 15 about 44%.

While the average results of 17 analyses of the effluent from bed No. 15 shows 0.9 parts of nitrates per 100,000 parts, it is noticeable that the nitrates gradually increase from 0.016 parts in May, 1900, to 2.41 parts in January, 1901; that they dropped to 0.91 parts in May, 1901, and have since gone down to 0.015 parts in July, 1901.

The effluent from bed No. 15 has been turned upon bed No. 16 at a rate of 730,000 gallons per acre and on bed No. 17 at a rate of 825,000 gallons per acre for six days in the week, with a much farther reduction of the ammonia and an increase in the nitrates, as shown by the tables of analyses.

Beds Nos. 16 and 17 are two small contact filters constructed by filling two galvanized iron cans, each 20 ins. in diameter and 6 ft. high, with filtering material; No. 16 with large pieces of coke, and No. 17 with cinders similar to those in bed No. 15.

WORK OF THE INTERMITTENT FILTER BEDS.

From Dec. 1, 1894, to Oct. 1, 1901, a total of 189,362,871 gallons of sewage was treated at the purification works, of which 186,604,895 gallons was applied to the 2.35 acres of intermittent filters, and 2,757,976 gallons (beginning March 8, 1900), to the contact beds Nos. 14 and 15. During the six years and ten months mentioned, 1,563 cu. yds. of poor (clogged?) sand was removed from the intermittent filters. This was at the rate of 8.25 cu. yds. per 1,000,000 gallons of sewage treated. All the sewage filtered, whether on the intermittent or the contact beds, was subjected to sedimentation or to septic action.

The total and average daily quantities of sewage treated during the fiscal year 1900-1, and its distribution by beds, for the whole year, is shown by Tables II. and III. Beds 1-4 (we understand) received sludge from the settling tanks from Nov. 14 to 19, 1900, and from Aug. 8 to Sept. 30, 1901, or for the few days between the two septic-tank periods, and after the last one. Beds 14 and 15 are the larger of the two sets of contact filters.

TABLE II.—Sewage Received and Treated at the Plant During the Year.

1900-1.	Sewage, galls. Total.	Sewage, galls. Avg. pr day.	1901.	Sewage, galls. Total.	Sewage, galls. Avg. pr day.
Oct.	3,641,093	117,467	May	2,758,187	88,974
Nov.	3,248,938	108,298	June	2,506,221	83,541
Dec.	5,261,488	169,725	July	3,862,666	124,602
Jan.	4,694,393	151,432	Aug.	3,552,252	114,589
Feb.	3,747,226	133,830	Sept.	3,930,936	131,031
March	3,428,837	110,608			
April	1,108,766	35,605	Total.	41,736,003	114,345

TABLE III.—Showing the Application of Sewage to Each Bed During the Year 1900-1.

No of bed.	Area (acres).	Sewage: Ordinary — No. of doses.	Sewage: Ordinary — Average quantity of dose, galls.	Sewage: Heavy — No. of doses.	Sewage: Heavy — Average quantity of dose, galls.	Total quantity on bed during year, galls.	Equivalent av'r'ge daily quantity per acre, galls.*
1	0.126	46	13,154	25	10,080	857,100	18,634
2	.132	42	13,828	23	10,560	823,680	17,097
3	.133	58	13,896	19	10,640	1,008,140	20,767
4	.123	61	12,804	18	9,840	958,170	21,341
5	.307	113	30,700	..	.....	3,469,100	30,954
6	.211	181	21,100	..	.....	3,827,380	49,693
7	.180	192	18,000	..	.....	3,456,000	52,595
8	.157	201	15,700	..	.....	3,155,700	55,066
9	.176	195	17,600	..	.....	3,432,000	53,428
10	.178	235	17,800	..	.....	4,183,000	64,382
11	.183	233	18,300	..	.....	4,263,900	63,830
12	.219	250	21,900	..	.....	5,475,000	68,490
13	.218	240	21,800	..	.....	5,232,000	65,750
14	.0054	282	2,157	..	.....	777,397	394,446
15	.0054	282	2,900	..	.....	817,456	414,814

*Figured on the basis of 365 days in the year.

Mr. Carpenter comments on the sludge problem, with and without the septic tank, as follows:

One of the difficult problems connected with all systems of sewage purification is the separation, treatment and disposal of the sludge contained in the sewage. Our experimental work with a septic tank furnishes interesting figures for comparison with the figures obtained from the treatment of sewage by sedimentation and intermittent filtration during previous years.

Six years' experience with settled sewage has given an average of 3.14 cu. yds. of sludge per 1,000,000 gallons of sewage treated, but recent figures made during the months of August, September and October, 1901, have shown that the amount of sludge has increased to 4.98 cu. yds. per 1,000,000 gallons of sewage, and that it weighs about 43.75 lbs. per cu. ft. This makes the weight of sludge now handled per 1,000,000 gallons of sewage about 2.87 tons.

The amount of sludge collected in the septic tank has averaged, during both trials, 5.126 cu. yds. per 1,000,000 gallons. The specific gravity of the sludge collected during the last experiment was 1.02, which would make the average weight of sludge that must be handled after treating sewage by the septic process 4.41 tons per 1,000,000 gallons. This latter sludge contains 79.11% of moisture and is more difficult to handle and much more offensive than the sludge which is raked from sand beds.

It is, however, only fair to say in this connection, that with a septic tank adapted to the needs of this method of sewage purification, this sludge could probably, with the addition of more sewage, be pumped to some disposal area instead of shoveled into barrows and wheeled from the tank as we were obliged to do. The comparison of the nature and amount of this sludge is, however, of value.

Sludge from settled sewage, dried on sand beds, resembles patches of tarred paper a square foot or more in area and can be readily raked from the surface of such beds in dry weather. Sludge from the septic tank is a muddy and offensive mass of matter, difficult to handle in barrows and probably difficult to dry out on any sand beds on which it could be pumped, unless it could be spread upon the sand in a thin layer.

Our experience has been, that during warm weather sludge from settled sewage turned upon sand beds is more easily handled and disposed of than the sludge from a septic tank.

It is probably true that a septic tank of greater depth than that in which we were obliged to perform our experiments might accomplish a greater reduction of sludge than we were able to obtain, but detailed and reliable data upon this subject seem to be lacking, at least in this country, where we were obliged to treat domestic sewage of considerable strength.

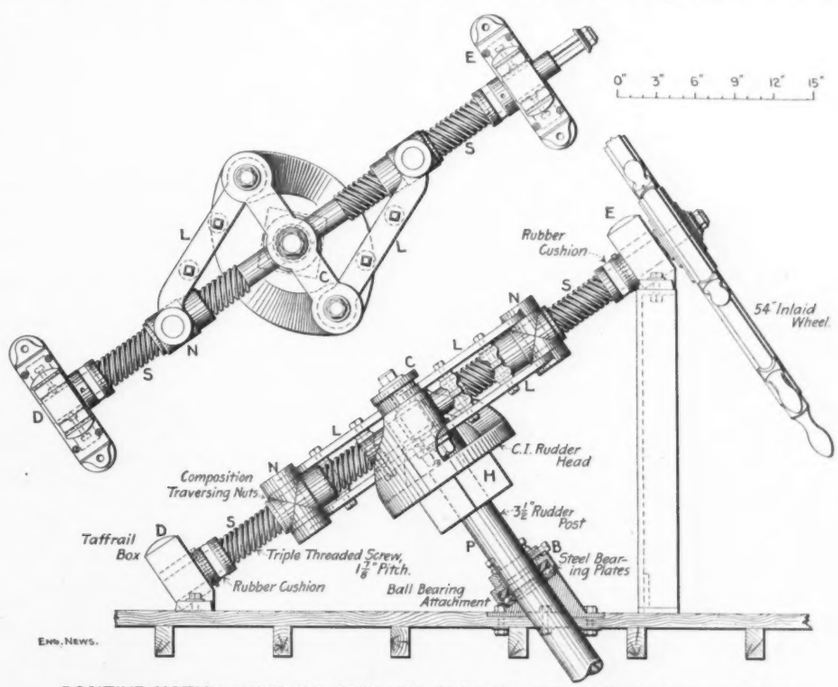
It seems probable that during the winter months, when the sludge from settled sewage will freeze on the surface of sand beds, will accumulate in considerable quantities and can only be removed after warm weather has arrived, the septic tank may offer a better solution of the sludge problem than such exposed beds of sand. We propose to experiment along this line the coming winter.

STEERING GEAR OF THE YACHT "METEOR."

The schooner yacht "Meteor," built for Emperor William II., of Germany, at the yard of Townsend & Downey, Shooter's Island, New York, and launched on Feb. 25, is fitted with a novel steering gear. This gear was designed especially for this vessel by Mr. G. D. Loud, of the Edson Mfg. Co., Boston, Mass., which constructed the mechanism and furnished it to the builders of the vessel.

The accompanying illustration shows the device in plan and elevation, and with a few explanatory words will make its action clear. The rudder post, marked P, is fitted at the top with a cast head, H, which has two horns projecting from its upper surface. A right and left-hand screw, S, is mounted in a fore and aft direction above the center of the rudder head, and on this screw traverse bronze nuts, N, connected by forged links, L, with the horns on the rudder head; the horns are connected by a crossbar, C. The screw, S, is directly operated by the wheel, W, manipulated by the helmsman. The arrangement is clearly seen in the plan. Turning the wheel moves the nuts, N, and these, through the links, L, turn the rudder head and post, with the attached rudder.

More in detail, the rudder post is of 3½-in. round steel; the cast-head, H, is formed with a 7-in. square offset on its lower face so that in case the gear should break, a wrought-iron tiller may be placed on the head for steering the vessel. The right and left screw, S, is 2⅞ ins. outside diameter, and has a triple thread of 1⅝ ins. pitch. The after taffrail box, D, is supported directly on the deck; the forward box, E, is raised on iron stanchions fastened to the deck. At the ends of the screw, S, are fastened collars, and against these rest cushioning rings of rubber, over which a loose brass collar is placed. The gear permits of a motion of the rudder to 45° either side of the center, and the rubber cushion is for the purpose of absorbing the shock when the nuts, N, reaches the outer ends of their travel. The weight of the rudder post, P, is carried on a ball bearing, B, at the deck. The bearing, or raceplates, between which the balls run are of steel, and the upper one is adjustable up and down by means of four screws passing through the cap of the bearing. The right and left screw, S, has a center bearing on the rudder head, being held in a box pivoted at the bottom in the head, P, and at the top in the crossbar, C.



POSITIVE MOTION STEERING GEAR FOR EMPEROR WILLIAM'S YACHT "METEOR."
Geo. D. Loud, Designer. Edson Mfg. Co., Boston, Mass., Builders.

The wheel, W, 54 ins. in diameter, is very elaborately wrought. Its spokes are of rosewood, the felloes of teak, and the rim of ebony. The Emperor's coat of arms is inlaid in the rim in gold and silver, and the wheel and spoke tips show variously the German cross and the German crown.

THE LUXEMBURG 277-FT STONE ARCH VIADUCT.

(With two-page plate.)

The longest stone arch bridge in the world is now being constructed over the deep valley of the Petrusse, in the Grand Duchy of Luxemburg, to connect the city of that name with its railway station on the other side of the valley. This bridge will have an arch span of 277.65 ft., which is 57.65 ft. greater than the span of the Cabin John arch at Washington, D. C., that has so long held the record as the longest span stone arch bridge in the world. It will, moreover, accommodate general street traffic, while the Cabin John arch carries an aqueduct with only a narrow footway on top. In addition to its great size, the new German bridge is of a particularly novel design. Briefly described, it consists of two parallel masonry arches separated by an open space of 19.41 ft., which is bridged over with a platform of concrete-steel construction. Together, the boldness and novelty of its design make the Luxemburg arch one of the most notable engineering works of modern times, and engineers will be interested in the account of its general details, which we abstract from "Le Genie Civil" of Jan. 18, 1902.

GENERAL DESCRIPTION.

The city of Luxemburg is located on a rocky plateau, surrounded on three sides by the precipitous gorges, some 130 to 165 ft. deep, through which flow the Alzette and its tributary, the Petrusse. The natural strength of its location led to the city being strongly fortified at a very early date, and every effort has been made to preserve its strength as a fortress. For this reason, when the first railway was built to the city, it was decided to locate the station on the opposite side of the valley, over a mile from the center of population. To give access to the city from the railway station a bridge was thrown across the gorge of the Petrusse. This structure was a stone arch viaduct 350 m. (1,148 ft.) long, with a roadway 8 m. (26.24 ft.) wide, and for a long time it has been inadequate to handle the constantly increasing traffic. To remedy this trouble it was proposed in 1875 to construct a second bridge, and a number of designs were presented for the new structure, some being for steel arches and others for stone arches. The last of these proposed structures was designed in 1897 by M. Rodange, Chief Engineer, Public Works of Luxemburg, and was a single span stone arch bridge. This design was put into the hands of M. Sejourne, who had previously executed the great arches of Larraur, 61.5 m.; Castalet, 41 m.; and Antoinette, 50 m., for elaboration. It is according to the final plans of M. Sejourne that the bridge is being constructed.

As will be seen from the illustrations, the new

bridge consists of a large center arch flanked on each end by a smaller approach arch. Spandrel arches, carried on the haunches of the main arch, support the roadway at those points. The main dimensions of the several arches are as follows:

	Meters.	Feet.
Main Arch:		
Span at springing line	84.65	277.65
Clear height	42.	137.76
Span at abutments	72.	236.16
Rise above abutments	16.2	53.23
Radius of intrados at crown	53.	173.84
Thickness of arch at crown	1.44	4.72
Thickness of arch at abutment	2.16	7.08
Width of intrados at crown	5.53	18.13
Width of intrados at abutment	6.12	20.27
Width of intrados at foundations	9.22	30.24
Rise from foundations to crown	31.	101.68
Space between arches at crown	5.92	19.41
Space between arches at foundations	2.25	7.38
Spandrel arches:		
Span	5.4	17.71
Thickness at crown	0.54	1.77
Thickness of piers at springing lines	1.08	3.54
Approach arches:		
Span	21.6	70.84
Thickness at crown	0.88	2.88

As has already been stated, and as is shown by the illustrations, the bridge consists of two parallel-arched structures, separated by an open space, which is bridged over by a thin platform. The advantages gained by this method of construction are: A material reduction in the weight and strength of arch center required, and a reduction of about one-third in the volume of masonry required as compared with a solid arch of equal width over all. By dividing the structure it was also possible to complete the arch ring for each half in a single summer. Thus the first half of the bridge had its centers struck on Oct. 26, 1901, and the present winter is being employed in getting everything ready for the masonry work of the second half, which is to be completed next autumn.

The principal structural features of the bridge are quite clearly shown by the drawings on our two-page plate. It will be noted, first, that the arch ring is built up of three ring courses. In constructing each of these courses it is divided into a number of segments separated by open spaces which are closed after the remainder of the course is completed. The object of this method of construction is explained as follows by the engineer: Despite the utmost care in its construction, a center for so great an arch is bound to assume a wavelike profile, with high points where the web members support the top chord and low points where, between panel points, the top chord members are unsupported. Hence fissures are inevitable in the arch ring at each point, where the top chord of the center is rigidly supported by the web members. If now a narrow artificial fissure, the full depth and width of the course, is left open at each point, where a natural fissure would be likely to occur because of the reason stated, the inevitable distortion takes place freely at these points without any rupture of the intermediate masonry, and, after the ring has finally adjusted itself, the openings themselves may be tightly closed by stone cut to fit the form assumed by them. A supplementary advantage gained by constructing each course in segments is that work may be carried on simultaneously at as many different points as there are different segments. Thus at the Luxemburg Bridge each ring course has been constructed from ten points of attack, and was completed in from eight to ten days. In the three courses of the arch ring there were 59 openings left to take up the distortion in the manner already described.

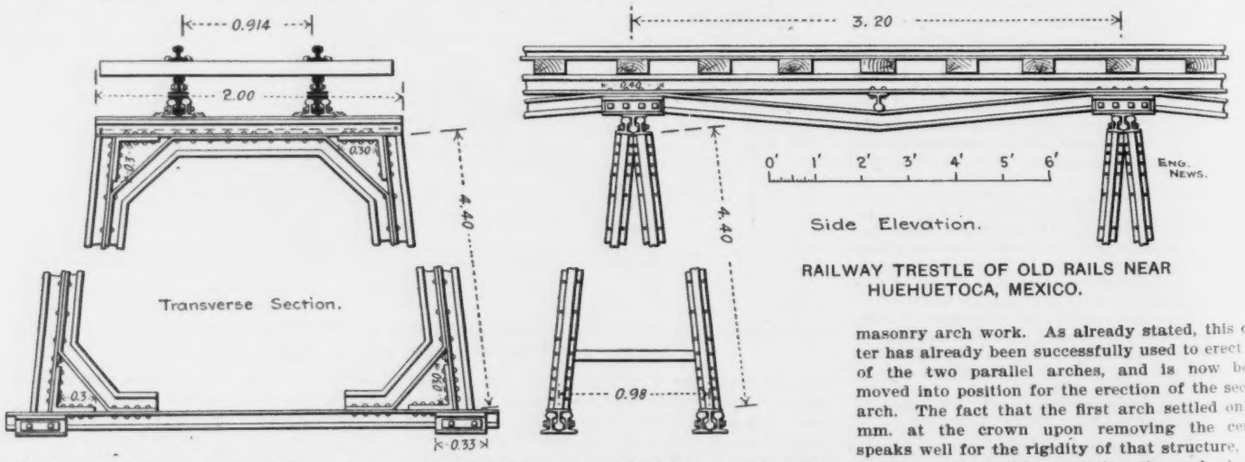
The material employed in the arch ring is cut stone from the Gilsdorf quarry, Luxemburg, having a crushing strength of from 1,200 to 1,500 kgm. per sq. cm. The intrados course is of smooth cut stone, and the upper courses are of hammer-faced stone. The face stones of the two lower ring courses have vertical faces, and the middle course overhangs the intrados course about 1-12 of the depth of the arch ring. The extrados course face stones have molded faces, and they overhang the middle course about 1-6 the depth of the arch ring. The preceding description of the arch ring construction refers only to that portion of the ring above the false abutment level. Below this level the three-course formation is discontinued, and the ring is composed of radial courses composed of a varying number of stones, as shown in the section, Fig. 3, on the two-page plate. The face stones of these lower portions or segments have a rather bald quarry face finish, with deep-cut arises. The same finish is used for the lower portions of the skewback piers. The upper portions of the skewback piers and the abutment piers are of dressed ashlar, with deep, rounded arises. The wing walls and the screen walls between the arch ring and the skewback piers are of uncoursed rubble. All the other masonry is smooth-dressed ashlar. For the main arch ring Vicat artificial cement mortar was used; for all the other masonry lime mortar was employed. The spandrel arches have single-course arch rings, with the face stones molded to conform with the three-course facing of the main arch ring, while the spandrel arch piers viewed in end elevation resemble columns with base, shaft and capital complete. Over the main span the sidewalk railing consists of heavy turned balusters carrying a massive stone hand-rail; over the shore spans a solid parapet wall takes the place of this open railing.

As will be seen from the drawings, the whole bridge between the end abutment piers is really made up of two parallel bridges spaced some distance apart, and identical in construction. These two parallel bridges are joined into a single structure for traffic by a concrete-steel platform about 22½ ft. wide and 1.15 ft. thick. This platform is shown in the sections, Figs. 4 and 5, which also show the broken stone filling on which are laid the roadway pavement and the sidewalks. Underneath each sidewalk the filling is laid on a waterproof coating of asphalt, which protects the arch masonry. The other details of the construction are quite obvious from the drawings.

METHOD OF ERECTION.

One of the objects of designing the Luxemburg Bridge in the form of two parallel structures was to facilitate the work of erection, by reducing the amount of falseworks required, and by dividing the masonry work into two separate tasks, each of which could be accomplished in a single season. The contract for erection was awarded to Fougerolle Bros., of Paris, France. As the city end of the bridge opened directly onto the city streets, it was necessary to locate all the workshops and yards at the opposite end, on the plateau, on which is located the railway station. A spur track from the railway line was built to the shops and yards. While the reception and storage of all material was of necessity confined to one end of the bridge, the work of putting this material in place in the structure had to be carried on simultaneously at both ends, and at numerous intermediate points. To meet these conditions it was necessary to design a structure which would enable the structural material to be taken from the storage yards and distributed at any point between the two ends of the bridge. Furthermore, this structure must be of such a design that it could be used to erect the second parallel arch after the first one was completed. The structure which was worked out by the contractors is shown by the drawings, Figs. 6 to 9 inclusive, on our two-page plate. It consists of a wooden truss bridge 7 m. (22.96 ft.) wide, supported on timber trestle towers and spanning the gorge directly over the first parallel structure. On the bottom chords of this bridge there was a floor system carrying a narrow gage railway track for material cars. On the top chords there was a track for a traveler constructed as shown by Fig. 10, which took the material from the narrow gage cars and lowered it to the arch masonry below. This timber structure required in its construction only 320 cu. m. of timber, and cost only 25,000 francs.

The structure, which called for the greatest skill and care in its design and erection, was, of course, the center on which the main arch span masonry was built. Like the service trestle, the center was designed and built so that it could be used first for one and then the other of the two parallel arches. Fig. 1 shows the construction adopted. Briefly described, a trussed arch center, tied together by steel cables, was mounted on two masonry piers or walls located inside the feet of the main arch. This structure required about 380 cu. m. of timber, and its weight was about 300,000 kgm. The extreme lightness of the center used is in some respects one of the most remarkable features of this remarkable example of masonry arch work. As already stated, this center has already been successfully used to erect one of the two parallel arches, and is now being moved into position for the erection of the second arch. The fact that the first arch settled only 6 mm. at the crown upon removing the center speaks well for the rigidity of that structure, and for the method of constructing the arch ring in segments, which has been described.



RAILWAY TRESTLE OF OLD RAILS NEAR HUEHUETOCA, MEXICO.

A RAILWAY TRESTLE OF OLD RAILS IN MEXICO.

We are indebted to Mr. B. F. Dickson, Assistant General Superintendent of the Mexican National Ry., for a blue print showing a trestle made of old rails and still in use upon one of the branches of that system. The drawing explains itself. Two old rails riveted together by the flanges form each of the four legs of the trestle, and three old rails, interlocked and connected by bolts and rivets, form the cross-members at the bottom and the top. Considerable labor has been expended in shaping the inner rail of each leg so as to form top and bottom angle braces.